SEISMIC DESIGN DECISION ANALYSIS

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SEISMIC DESIGN DECISIONS FOR THE COMMONWEALTH OF MASSACHUSETTS STATE BUILDING CODE

by

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This report focuses attention on questions of estimating seismic risk in Massachusetts, determining a design earthquake, developing appropriate soil factors, and determining acceptable risk. Aside from documenting the decision process of the Seismic Advisory Committee, this report also attempts to evaluate the role of Seismic Design Decision Analysis (SDDA) methodology and the loss estimates provided by the research project in formulating recommended design criteria. The overall aim of this research is to develop data and procedures for balancing the increased cost of more resistant construction against the risk of losses during future earthquakes. To achieve this objective SDDA has provided a coherent structure for technical inputs from seismology and engineering research to the public policy process for earthquake hazard reduction.

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ABSTRACT

The objective of the Seismic Design Decision Analysis project has been to provide a coherent structure for the technical inputs from seismology and engineering research to the public policy process for earthquake hazard reduction. In developing a methodology for seismic design decision-making, a particular effort has been made to work with engineers, buiilding officials and public bodies to learn how such data and results can be used as a basis for making decisions about seismic design requirements.

In 1972 the Commonwealth of Massachusetts created a State Building Code Commission charged with the promulgation of a uniform State Building Code. In July 1973 a Joint Committee on Seismic Design Criteria was formed by the American Society of Civil Engineers/Massachusetts Section and the Boston Society of Civil Engineers to review seismic design requirements in effect in the Commonwealth. The recommendations of this Seismic Advisory Committee were eventually incorporated into the Massachusetts State Building Code promulgated by the State Building Code Commission in 1975.

The deliberations and concerns of the Seismic Advisory Committee are of interest for several reasons. First, they represent a unique effort to develop seismic design provisions particularly suited to an eastern seismic area. Second, they offer general insight into the decision-making process relating to seismic building regulation. Third, through the direct participation of the principal investigaors of the SDDA project, the development of the Massachusetts Seismic Provision represents a first partial application of the SDDA methodology to an actual public decision.

This report reviews the background for the work of the ASCE/BSCE Joint Committee on Seismic Design Criteria and the considerations taken by the committee in formulating its recommendations. Attention is focused on the questions of estimation of seismic risk in Massachusetts, determination

of a design earthquake, development of appropriate soil factors and the determination of acceptable risk. Aside from documenting the decision process of the committee, this report also attempts to evaluate the role of the SDDA methodology and the loss estimates provided by the research project in the formulation of recommended seismic design criteria.

PREFACE

This is the thirty-second in a series of reports under the general title of Seismic Design Decision Analysis. The overall aim of the research is to develop data and procedures for balancing the increased cost of more resistant construction against the risk of losses during future earthquakes. The research has been sponsored in part by the Earthquake Engineering Program of NSF-RANN under Grant GI-27955. A list of previous reports follows this preface.

This report is a documentation and discussion of the decision process of the ASCE/BSCE Joint Committee on Seismic Design Criteria in formulating the seismic provisions of the Massachusetts State Building Code. The deliberations of the committee are described and the role of Seismic Design Decision Analysis methodology is evaluated.

The author is a Research Associate in Civil Engineering at M.I.T. Professor Robert V. Whitman, the principal investigator of the overall study had a key role in initiating the work of the Joint Committee on Seismic Design Criteria and has provided detailed information on the committee's work and other valuable suggestions for this report.

The author is indebted to Dr. Howard Simpson, Chairman of the Seismic Advisory Committee, and Mr. Norton Remmer, Commissioner of Code Inspection in the City of Worcester (formerly Technical Director of the Massachusetts State Building Code Commission) for their generosity in providing detailed information on the actions of the Seismic Advisory Committee and the Massachusetts State Building Code Commission. Professor Whitman, Dr. Simpson and Mr. Remmer have also generaously taken the time to review an early draft of this report.

The author also appreciates information provided by Mr. Herbert Isenberg, Architect, Boston,Massachusetts; Mr. Francis Harvey, Consultant Engineer, Worcester, Massachusetts; Dr. Frank J. Heger, Consulting Engineer, Cambridge, Massachusetts; and Prof. Allin Cornell, Department of Civil Engineering, M.I.T.

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INTRODUCTION

The Seismic Design Decision Analysis Project has pursued three long-range goals:

1. To develop reliable and acceptable data concerning the tangible costs and benefits of designing for increased seismic resistance;

2. To develop probabilistic models for analyzing and comparing the costs and benefits of various strategies for mitigating the consequences of expected or anticipated future earthquakes.

3. To work with engineers, building officials and public bodies to learn how such data and results can be used as a basis for making decisions about seismic design requirements.

In the development of an analytical decision aid, it is of critical importance to understand the context of decision-making, to know who the decision-makers are and the constraints under which they work. This report is in the form of a case study. It attempts to reconstruct, and to some extent, analyze, the decision-making process which led to the insertion of the seismic design provisions in their adopted form in the 1975 Massachusetts State Building Code.

This particular code decision is of interest for several reasons. It represents one of the first serious considerations of building regulation for seismic design in the eastern United States. The Massachusetts Seismic Code decision is also interesting as an example of the engineering profession's capacity to integrate public responsibility in technical decision-making as a model for other eastern metropolitan areas. Furthermore, because the principal investigators of the SDDA project were active participants in the ASCE/BSCE Seismic Advisory Committee, the code decision represents the first trial of preliminary SDDA results.

How are decisions made on Seismic Design Criteria?

This report attempts to trace the thinking which lies behind the Seismic Advisory Committee recommendations, and to see to what extent the final result reflects the influence of SDDA research. An effort is also made to evaluate the role played in the decision process by risk assessment and cost/benefit analysis as opposed to intuitive judgment.

In tracing the Seismic Code decision process, attention will be focused on

a) the problems posed;
b) the decisions made
c) the basis for those decisions.

It is reasonable to ask if the resulting decisions are correct and how we can know if they are correct. Because of the uncertain nature of seismic activity and the remaining ambiguities in the act of seismic design, it is difficult to define what would constitute right decisions in the case of seismic regulations. So long as no earthquake occurs seismic regulations will appear superfluous, but in the event of an improbable but possible major earthquake the same seismic regulations may appear inadequate. Because of this difficulty, we may more usefully discuss what constitutes a good decision rather than a right one. Good decisions are determined by the methodology on which they are based. A good decision will make the best possible use of currently available information and understanding. This means that to evaluate the quality of such a decision we must examine the thought process on which it is based.

It is also fair to inquire to what extent code decisions can be regarded as purely technical problems. At what point do they become the appropriate subject matter of political debate and decision processes? To what extent do technical groups make political decisions, and to what extent do social and political considerations complicate technical decision-making? The establishment of seismic building regulatory standards is a particularly complex case of the interaction of objective technical judgment and subjective political considerations.

The study of an actual seismic code decision process in a "real world" context, socially, economically and politically, should provide some useful insight for committees which are now approaching similar decision points regarding the adoption of seismic provisions.

Hopefully the experience of Massachusetts can be instructive for other eastern jurisdictions now considering seismic provisions

CHAPTER I: BACKGROUND

1.1 Building Codes in Massachusetts

Following the Great Boston Fire of 1872, the first Boston Building Code was promulgated in 1874. This building code was originally enforced by the police department, until the City of Boston established a Building Department, which took over responsibility for building regulation in the city. Since 1913, there have been initiatives to develop a uniform building code for Massachusetts.

In 1942, in the Coconut Grove night club fire, 491 people were killed. This disaster lead to major revision of fire codes. Amendments were made to Boston City Code in 1943, and a State Commission in 1945 recommended that a uniform building code be developed by the State Board of Standards, to be adopted voluntarily by local communities. The Board of Standards Code was finished in 1949 but even by 1960 there were few communities in the Commonwealth which referred to the voluntary code.

Gradually over time there developed a body of relatively uncoordinated minimum standards issued by various state agencies. There was no compulsory State Building Code, as such; Section 3 of Chapter 143 of The General Laws of Massachusetts allowed municipalities to promulgate their own building codes so long as they met certain minimum standards established by the state. Such compulsory standards established at state level included regulations governing: schoolhouse construction, egress for places of public assembly (more than 400 persons), apartment houses, plumbing, electrical work, elevators, and gas piping.

The state level involvement in building code issues was the province of the Department of Public Safety. The Department of Public Safety was the only unified enforcement organization, including state police, fire marshalls, and building inspectors. Operating under the authority of Chapter 143, The Department of Public Safety maintained a force of 32

State Building Inspectors. They approved plans for the public assembly buildings, schools and other special cases in which the state exercised authority. Each local municipality had its own building code and the local authority was the primary enforcer of the local code as well as the Board of Standards regulations.

The local jurisdictions within the Commonwealth were free to adopt their own codes subject to the provisions of Chapter 143. The limited state regulations for special occupancies which were enforced by the State Department of Public Safety were based on a slightly amended version of the 1970 edition of the BOCA (Building Officials Conference of America) code. However, because this was not extensive and only applied to special occupancies on a uniform basis, it did not imply a uniform state building code.

Aside from the provisions and regulations issued by the state governing electrical, plumbing, egress, gas piping and elevators, there were also myriad confusing, and sometimes contradictory,regulations issued by various agencies and commissions in the Department of Public Safety and elsewhere in the state government. In 1970 twenty-seven separate state agencies independently issued regulations affecting construction in the state. Problems of coordination were monumental. Complexity and inconsistency of regulations and regulatory bodies made the building regulatory function in the state chaotic. Builders, developers and building designers suffered particularly from the uncertainty of the regulatory process.

In 1968 the state established the Department of Community Affairs and in 1970 in the Division of Community Services, the Office of Code Development was established to review model codes. Pressure was brought to bear on the state government by developers, large contractors, professionals and especially system builders to review and update building regulation practices in Massachusetts.

The interest of industry in code uniformity has been almost entirely directed to the elimination of hindrances to development of large markets for standardized building products and standardized designs. There has been limited infighting in those cases where a particular building material sector, such as steel or masonry, feels that its competitiveness will be affected by particular code measures.

Considerable encouragement for code modernization also came from the federal government through the Department of Housing and Urban Development. HUD and the building industry were in favor of harmonizing local building regulations and the development of uniform state building codes in order to reduce what was seen as a serious obstacle to the development of large scale markets for prefabricated building components and the general technological advance of the building industry.

The role of HUD, like that of the construction industry, has consistently been to encourage greater standardization of building regulations. However, because building regulation, as a police power, is constitutionally reserved for the states and traditionally delegated to the local level, the federal government is not in a position to promulgate or enforce building codes. HUD has been known to impose its own standards by withholding funding until a municipality modified its code to agree with HUD concepts. HUD has also tried, through funding background studies and other means of incentive, to promote a harmonizing of building standards.

As a sizable proportion of total construction is built with some form of federal subsidy (an estimated 30% is directly federally funded, and an estimated 60% has federally guaranteed financing), the federal government has been able to exercise considerable leverage on state and local regulations.

Other federal agencies have been able to exert the influence of example through development of standards of seismic, wind and flood plain design which apply to federal construction and provide a standard of comparison for local codes.

Efforts have also been made by federal agencies to influence construction practice through the establishment of minimum property standards as conditions for participation in federally subsidized mortgage and insurance programs.

During 1970 and 1971 there were numerous consultant studies commissioned by the state on code questions. In December of 1970 a document, Reports Relative to the Development, Administration, and Enforcement of Building and Housing Codes, was submitted to the Department of Community Affairs. This document was a synthesis of reports and inquiries on code policy in Massachusetts. It was prepared with support from the U.S. Department of Housing and Urban Development, and the assistance of the Joint Center for Urban Studies of the Massachusetts Institute of Technology and Harvard University. The report was prepared in order to:

1. Review the building regulation function as it is carried out in cities and towns of the Commonwealth;
2. Describe some of the shortcomings of current practice;
3. Identify the special set of problems, largely brought about by the present system of regulatory building construction, that confront any attempt to introduce modern systems technology into the building process;
4. Gauge the impact of building regulation policy in the state on the nascent manufactured housing industry;
5. Identify a number of approaches to code reform taken by the government throughout the country;
6. Recommend state action to remedy the most injurious faults of the current regulatory system.

The studies found that only one third of the cities and towns of Massachusetts had full-time professional building officials. One third had only part-time building inspectors and one third had none. On the state level, 27 different agencies independently regulated and established standards for various aspects of building.

The study found that: "The regulation of the building industry had as its impetus the protection of life and the preservation of life safety" but that "Today, consumers of housing and users of buildings demand more: a measure of assurance that they are getting their money's worth." The report intended to examine state actions which "would expedite the provision of those structures vital to the Commonwealth's progress."

The main thrust of the report was to question the "logic of localism" and to recommend that the state reclaim its responsibility in the field of building regulation. This responsibility was seen to be best filled through promulgation of a mandatory uniform state building code. The primary intention of the uniform code was to unencumber the building industry from the maze of conflicting and inconsistent building regulations. The code intended to open the field for large scale building industry development, not to increase restriction. The uniform code was to develop uniform dimensional standards on a state (and national) level to allow for creation of large markets for prefabricated building systems and components. Public interest was in taking advantage of the hoped for savings of industrialized construction rather than increased public safety. The report, which was issued in April 1971, served as the basis for the enabling legislation for the establishment of a state building regulatory agency and the promulgation of a mandatory uniform state building code.

In response to the studies of the Department of Community Affairs and incentives provided by the federal government (HUD), the General Court of the Commonwealth passed enabling legislation (St. 1972, C.802) for the establishment of a State Building Code Commission, the promulgation of a State Building Code, and the establishment of a building regulatory system. This legislation passed in 1972 created provisions for a single statewide building code, which would replace all existing building codes in the state and would mandatorily apply to every jurisdiction within the Commonwealth. The statewide code was required to be promulgated on July 1, 1974, and to become legally effective on January 1, 1975.

In hindsight it has been argued that the enabling legislation was based on rather vague objectives and has led to inadequate solutions. The re-organization of administrative responsibilities has created new ambiguity and possibility for confusion. The Department of Public Safety which had formerly been in charge of state building inspection functions was unwilling to condone a revision of the building regulatory system which would decrease its role. This opposition to revision of the inspection and enforcement system has led to a compromise solution which has several unfortunate aspects.

Building officials in Massachusetts have typically been political appointees. In the course of the building code reform the political process has not seen fit to divest itself of this channel for political patronage. While some building officials have had long tenure over many administrations, there has been a general reluctance to give building officials a recognized civil service and professional status.

The State Building Code Commission

The State Building Code Commission was established in February, 1973. It was mandated to promulgate a uniform state building code by July 1, 1974. It is made up of eleven commissioners who are appointed by the Governor. Two members are designated ex officio. They are from the Department of Community Affairs and the Department of Public Safety. The State Fire Marshall also sits on the commission. The other eight members are political appointees.

The first job of the Commission was to promulgate the code. The Massachusetts State Building Code was under preparation during 1973 and 1974 and was promulgated on January 1, 1975. Like the previous Boston City Code, it is largely based on the Building Officials Conference of America (BOCA) Basic Building Code/1970, Fifth Edition, and the BOCA Basic Building Code Accumulative Supplement 1973.

The Commission is further charged to maintain the Code and keep it up to date. The mandate (Chapter 802) states that the Commission is the sole authority for building codes in the state. A majority vote of the Commission can change the code. There must be public hearings and Commission action must follow within 90 days of the public hearings. All material must be available for the public before the hearing. There are regular mandated public hearings on the entire code in May and October each year.

1.2 Seismic Provisions in Massachusetts

As of 1970 it could reasonably be said that earthquakes were virtually ignored in the design of conventional buildings in the Commonwealth. Though records were available documenting the major earthquakes of 1727 and 1755, which affected large areas of eastern Massachusetts,it was not common knowledge for the public, public policy makers or even the engineering community that Massachusetts was in a potentially acitve seismic area. However, knowledge and understanding of the seismic activity of the area has been increasing.

Two important inputs to the broader understanding of the seismic risk of New England were provided by the geological and seismological studies carried out by the Atomic Energy Commission for siting of nuclear power generating plant facilities and the development of a new seismic risk map of the United States by Dr. S.T. Algermissen of the U.S. Geological Survey.

In a paper presented to the 4th World conference on Earthquake Engineering in January, 1969, Dr. S.T. Algermissen of the U.S. Geological Survey proposed a new seismic risk map for the United States. Though the Geological Survey gave Algermissen permission to publish the paper including the map of proposed seismic zones, it did not give the map any official sanction.

The paper of which the map was a part made clear that the risk of earthquake occurrence in the East was very different from that in the West. However, the map is widely known outside of its context in the Algermissen paper. This map places areas around Boston and Charleston in the same category as California (Zone 3). While this suggestion taken out of context is misleading, it does indicate that earthquakes of intensity VIII or greater - involving at least fall of walls and chimneys, damage to many buildings and collapse of some buildings - must be expected in at least some east coast metropolitan areas. On the original map Zone 3 was defined as an area where MMI$\geq$VIII must be expected. However, there is a considerable difference between MMI VIII and MMI IX. There would have been good grounds for subdividing Zone 3 to clarify this distinction.

The Algermissen map used standard lists to establish seismic history. It has been argued that these lists represent an inflation of actual experienced seismic events.

The 1970 edition of the Uniform Building Code incorporated the revised Seismic Zone Map which had been prepared by Dr. Algermissen. This meant that the 1970 UBC suggested that buildings in Boston, Memphis and Charleston should meet the same earthquake design requirements as buildings in Los Angeles and San Francisco. In the eastern areas, which had been designated Zone 3 on the basis of controversial historical evidence, feeling was that this represented an overestimation of risk and implied an undeserved penalty. Everyone agreed that it is wrong to have the same earthquake design requirements for Boston as for San Francisco. However, it was not readily clear whether the requirements in San Francisco should be raised or whether those in Boston should be reduced.

Arguments were put forth suggesting that the historical data on which the Algermissen map was based have incorporated an upward bias in evaluating pre-instrumental seismic events. It was argued that the epicentral intensity of the 1755 Cape Ann earthquake was certainly not MMI IX and only marginally MMI VIII.

All of this contributed to the controversy surrounding the seismic zoning of eastern Massachusetts and suggested a reevaluation of the historical basis of the Algermissen map.

There was widespread feeling within the engineering profession that overly stringent seismic design criteria would add significantly to design and and construction costs and that they would require unnecessary adjustments in standard design procedures.

However, even in this somewhat skeptical climate efforts were made to deal responsibly with the emerging awareness of seismic risk. Two cases are worthy of note. At the state level, the first recognition of seismic risk appeared in a 1971 revision of the provisions for schoolhouse construction. The state board concerned with schoolhouse safety required that earthquakes be considered. However, the revision was based on a slightly amended version of the 1970 BOCA code which means that the seismic provisions were relatively weak.

The second case is that of the Building Code of the City of Boston. Under the direction of Building Commissioner Richard Thuma, the City of Boston initiated an effort to write a modern building code. The city commissioned Francis Harvey, a consulting engineer, and Herbert Eisenberg, an architect, to draft the new code. The initial draft of the code was the subject of an extended series of City Council hearings. The provisions were taken up chapter by chapter. The code to a large extent was based on BOCA, with inputs from the New York City code and various others. The Subject of earthquakes was not dealt with in the initial draft.

At that time, the issue of seismic design standards had not developed much beyond a general sense on the part of local engineers that Boston did not warrant the same level of seismic regulation as San Francisco but that some regulation may be needed.

Commissioner Thuma consulted Dr. Howard Simpson, Chairman of the Structural Section of the Boston Society of Civil Engineers. In answer to an inquiry as to whether Boston should be classified as Zone 2 or Zone 3, Dr. Simpson responded that it was not likely that the community would accept the same level of regulation as California (Zone 3) because the economic impact would be too great. Dr. Simpson recommended that the best that could be hoped for would be Zone 2. On the basis of this consultation and further consultation with Francis Harvey, Commissioner Thuma was able to make the decision to place Boston in Zone 2.

Though this signified a reduction of the zone designation according to the Algermissen Map and 1970 UBC, it was still a positive change as it introduced seismic considerations for the first time in the Boston Building Code.

The Building Code of the City of Boston was promulgated July 1, 1970, with the following section on earthquake load:

> "Section 719.0 Earthquake Load
>
> All structures except one (1) and two (2) family dwellings and minor accessory buildings shall be capable of safely withstanding the lateral forces prescribed for Zone 2 in Reference Standard RS 7-12."

Reference Standard RS 7-12 was ICBO 1967 Uniform Building Code, Vol. 1, Section 2314, "Earthquake Regulations."

This minimal coverage of seismic design has been criticized for its vagueness. It was intended that the provisions would be interpreted by informed engineers. It was not intended that one would design only for lateral force levels, but for ductility also. The Boston code did not spell out ductility provisions; however it was meant to present only minimum requirements, and not necessarily all minimum requirements.

The result is that some engineers designed for ductility and others did not. Unfortunately pressure was put on conscientious engineers by clients who threatened to take their business to those engineers who did not increase the cost of construction by including measures to increase ductility.

In summary, by 1972, when the new mandatory state code was being prepared for promulgation, the situation in Massachusetts was this: the state had generally ignored any concern for seismic design, except in Boston. The practice throughout the state was to assume that Zone 2 applied, although generally very few design provisions were applied.

The 1970 Uniform Building Code adopted the Algermissen Seismic Risk Map which placed Boston in Zone 3. If Boston updated its code to reflect the change in the map, it would mean that Boston was subject to the same requirements as Los Angeles. At the same time, if the traditional direction were taken, the state Department of Public Safety would amend its own code to reflect the forthcoming changes in the BOCA code dealing with seismic design. Those changes would classify the state as Zone 2 but incorporate more detailed design provisions approaching more closely those of the Uniform Building Code.

All these considerations became somewhat academic, however, because of the impending mandatory statewide code. It was the newly formed State Building Code Commission that would have to deal with the problem of a statewide policy on seismic risk and design.

1.3 The Seismic Design Decision Analysis Project at M.I.T.

In 1971 research work commenced in the Department of Civil Engineering at M.I.T., under the direction of Professor Robert V. Whitman on an NSF (RANN) sponsored project entitled "Optimum Seismic Protection for East Coast Metropolitan Areas." The long-range objective of this project was to develop regulations which would provide an optimum level of earthquake protection for new multistory housing in eastern metropolitan areas. The short-range objective of the study was to develop necessary analytical procedures and basic data and to validate use of those procedures in a study of the Boston Metropolitan area.

The research was carried out over the following two years. Research work was centered on five steps:

1. Evaluation of the earthquake threat in terms of response spectra applicable for different return periods and different soil conditions.
2. Analysis of structural response and resistance of typical multistory housing units, considering both overall resistance and design details.
3. Development of a methodology for risk analysis.
4. Studies which related design criteria to cost of repair necessitated by various earthquake intensities.
5. Assembly of results into seismic risk analysis and determination of the optimum strategy for providing earthquake protection.

As the results of this research project became available, Professor Whitman and the other principal researchers made an effort to disseminate these results to potential users in the local engineering community and to decision-makers concerned with seismic safety. In February and March of 1973 two presentations were made to the Boston Society of Civil Engineers giving the preliminary results of the NSF (RANN) sponsored research project. The presentation was made by Professors Whitman, Cornell and Vanmarcke. Attendance at each session was about 150. This attendance figure reflected the growing concern and interest in the engineering community for the problems of seismic safety in Boston and the rest of Massachusetts. These presentations also played a key role in focusing attention on the problem of seismic building code provisions while presenting a potential rational methodology for their development.

The M.I.T. research project was continued in 1973 under the title of "Seismic Design Decision Analysis for Eastern Metropolitan Areas. (Both the first phase and continuation are now referred to as Seismic Design Decision Analysis or SDDA.) The continuation project was directed to extension and expansion of the original objectives. It also addressed the problems of interaction with potential users of the SDDA methodology and implementation.

The planned implementation of SDDA was to be carried out by first working with professional engineering groups and then with building officials. Through the BSCE presentation and other similar dissemination efforts local professionals were informed that M.I.T. had been conducting a study of the costs and benefits inherent in different seismic design requirements. This study would not lead by itself to specific recommendations. Rather, the aim had been to assemble the basic input data required for any assessment of costs and benefits, and to develop an approach whereby engineers, building officials and the interested public could systematically arrive at a suitable balance between cost and risk.

A first step toward implementing seismic design decision analysis was a detailed review of seismic design provision in the Boston Building Code. Discussions were held with the leadership of the Boston Society of Civil Engineers and the Massachusetts Section of the American Society of Civil Engineers concerning the appointment of a committee of local engineers to undertake such a review. Professor Whitman and SDDA staff suggested that a possible scenario for the first year of the review might be:

1. Review, modify and finally endorse the analysis of initial cost increments and tangible expected losses.
2. Review analysis of intangible costs and develop a policy statement regarding the importance of such losses.
3. Recommend (if appropriate) changes to the present seismic design requirements.

It was considered that M.I.T could aid the committee's efforts by providing a framework for the review and by undertaking staff studies. The committee's efforts would in turn aid M.I.T.'s research effort by providing reactions and suggestions concerning the seismic design decision analysis methodology. Once the local professional community reached a consensus concerning desired changes then efforts could be turned to work with building officials and the local political community.

In summary, the background of the seismic code development for the Massachusetts State Building Code includes an interaction of several parallel lines of activity. First, the process of code revision and development for the promulgation had gotten under way for reasons quite apart from seismic risk. Second, the expanding concern for seismic risk by federal agencies such as the U.S. Geological Survey and the National Science Foundation contributed to the development of some controversy over the need for seismic regulations in Massachusetts. Third, the Seismic Design Decision Analysis project at M.I.T. had begun collecting data and developing a methodology which centered on facilitating public decisions on adoption of seismic provisions.

In the spring of 1973 these three factors combined to initiate the seismic code development activity in Massachusetts.

CHAPTER 2: THE ASCE/BSCE JOINT COMMITTEE ON SEISMIC DESIGN CRITERIA

2.1 Growing Awareness of Seismic Design Issues

The new Building Code of the City of Boston which included the UBC lateral force requirements for Zone 2 was adopted by the City Council and became effective July 1, 1971. Many builders objected to the new requirements, while others, including engineers and researchers, did not think they were strigent enough. Controversy continued as a background issue as the activities to establish a uniform state building code neared their conclusion. The SDDA presentations to the BSCE focused further attention on the seismic issue and it became clear that any effort to influence the level of seismic provisions in the state would have to be directed toward the new State Building Code Commission and the new uniform state code.

On May 2, 1973, Professor Whitman wrote to Mr. Max Sorota, President of the Boston Society of Civil Engineers and to Mr. Ronald Hirshfeld, President of the American Society of Civil Engineers, Massachusetts section, to recommend the formation of a committee to review the seismic design requirements currently in effect in the Commonwealth of Massachusetts and its towns and cities, and to recommend - if appropriate - changes in these requirements. Professor Whitman suggested that such a committee should play a role in Massachusetts similar to that played by the Seismology Committee of the Structural Engineers Association of California with regard to seismic design provisions for cities in California. The letter further stated that the SDDA project would stand ready to provide a significant level of staff support for such a committee. It is mentioned that the committee should be formed as soon as possible and that it should expect to work intensively for one or two years in order to provide a set of conclusions and recommendations. Suggested sub-committees included:

Seismic risk

Soil effects and foundation design

Structural analysis and design requirements

Non-structural design requirements.

The letter closes with the suggestion that the committee be formed as a joint effort of the BSCE and ASCE.

2.2 ASCE/BSCE Initiative

The response to this approach from both Mr. Sorota (BSCE) and Mr. Hirshfeld (ASCE) was very positive. Dr. Howard Simpson of Simpson, Gumpertz and Heger, consulting engineers in Cambridge, was asked to serve as Chairman of a Joint Committee on Seismic Design Criteria. By July 27 the Committee was formed and its membership complete.

It was announced that the Committee would provide input to the State Building Code Commission and provide a focus in Massachusetts for the continuing evaluation of seismic design criteria in light of new research and experience. The membership of the Committee was selected from the membership of the ASCE Massachusetts Section, BSCE, and selected liaison members from governmental agencies and major local engineering firms. The disciplines represented on the Committee included structures (ST), soils (SM), and seismology (SY).

<u>JOINT COMMITTEE ON SEISMIC DESIGN CRITERIA</u>

Massachusetts Section, American Society of Civil Engineers;
Boston Society of Civil Engineers

Dr. Howard Simpson, Chairman (ST)
Simpson, Gumpertz and Heger

Charles A.J. Theodore (Liaison)
State Building Code Commission

George H. Brattin (ST)
Portland Cement Association

John Brennan (ST)
LeMesaurier Associates, Inc.

Dr. Kenneth Leet (ST)
Northeastern University

Reverend Daniel Linehan (SY)
Weston Observatory

Donald E. Reed (SM)
Haley & Aldrich, Inc.

Maurice A. Reidy, Jr. (ST)
Maurice A. Reidy Engineers

Gonzalo Castro (SM)
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M.I.T.

Stanislaw J. B. Gawlinski (ST)
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The initiatives taken by the BSCE and ASCE Massachusetts Section, in setting up the Joint Committee on Seismic Design Criteria is not unique. The engineering profession has demonstrated its willingness through professional organizations to take on such public responsibiltiy on numerous occasions. However, it may be instructive for other seismic areas of the East to know that the development of seismic provisions for Massachusetts began with an independent initiative of local professional engineering groups.

2.3 Mandate of the Committee

The Committee was established on a voluntary basis without outside support. The Committee was, in fact, subsequently recognized as an official advisory committee to the State Building Code Commission. Over the period of the next 15 months, the Committee developed the basic seismic provisions adopted by the Commission.

The State Code Commission established as its first priority of responsibility in all code considerations the protection of life safety. In the case of code requirements to resist seismic effects, there was a necessity to establish a fundamental understanding of risk to apply to the concern for life safety. To start with, it seemed apparent that Boston should be distinguished somehow from Los Angeles in terms of implied risk and threat to life safety. Throughout the rest of the state, which fell into Zone 2, it was also necessary to obtain an understanding of what was implied by design and design requirements.

The question that naturally arose was whether the basis of design requirements and the zone designation, and consequently the total lateral force applied, were biased more towards economics rather than life safety. The economic issue represented a delicate problem. It was impossible to ignore the potential impact on construction costs that provisions would have at a time when the construction industry in Massachusetts was severely depressed. It was necessary to ensure a defensible balance between what appeared mandatory for life safety and what was reasonable as an increased construction cost. It is important to note that the primary concern here relative to economics was not what the threat of an earthquake represented in terms of regional impact or financial burden on the owner or the community subsequent to the event; it was the financial penalty imposed on construction and its effect on developing the economy.

In order for the Code Commission to assess their responsibility in readily comprehensible terms, it was necessary to translate the concepts of risk and return period in a way that would relate them to the public safety mandate of the Commission. The structural engineer on the Commission, Mr. Charles Theodore, and the technical director, Mr. Norton Remmer, sat with the Committee on Seismic Design Criteria and transferred the concepts and concerns between the Commission and the Committee. It was within the context of the apparent risk for Boston and the state, as presented to the Commission, that the Commission was able to explicitly establish the primary concern of life safety and the objective of minimizing the financial burden in construction.

The ASCE/BSCE Joint Committee on Seismic Design Criteria has now been officially recognized as the Seismic Advisory Committee of the Massachusetts State Building Code Commission. The Seismic Committee joins a range of other advisory committees which have been organized to support the work of the Commission. Other advisory and consulting committees of the Commission include:

Live Loads Committee

Construction Materials Safety Board

Massachusetts Construction Industries Board (which licenses concrete technicians and laboratories)

Fire Prevention Board

Massachusetts Hospital Association

Technical Code Council

The Commission has recognized the need for broad access to information and expertise in a range of specific fields. Though knowledge and understanding in many of these fields is expanding at a rapid rate, information is difficult to process and apply to a problem so technically and socially complex as code development.

Building codes have developed historically as an accumulation of unrelated responses to specific problems (which have usually been recognized in the context of some catastrophic failure). Codes do not represent a systematic approach to the building safety problem as a whole. They are largely an accumulation of ad hoc responses to limited problems tempered by political and social values which are often inconsistent with the present.

Because the code cannot be seen as a coherent system and there is no well developed technology of building regulation, it is difficult to formulate clear and specific questions for consultants. With both a political and a technical function, a body such as the Code Commission faces a very difficult task in defining priorities and objectives. There is often risk of conflict between the ideal of an abstract order and consistency in the code and the distraction of adjustments and exceptions to satisfy immediate political objectives.

The ability to get good advice is dependent on the ability to formulate clear questions. The Seismic Advisory Committee in beginning its work was primarily concerned with the clear formulation of the questions it sought to answer.

The Massachusetts State Building Code Commission was very fortunate to have access to the talent assembled on the Seismic Advisory Committee. Certainly the Boston area with its many universities and research facilities has unique resources. The Committee had access to federally sponsored research projects and local universities and the expertise of local engineering firms with worldwide experience in a range of specialized areas, including nuclear power plant site analysis and construction. Access to this information and talent may not be typical for other eastern jurisdictions facing similar decision on seismic design. For this reason it may be of value to extract as much experience as possible from the deliberations and decisions resulting from this process. It has been mentioned that many jurisdictions, especially those with limited resources, are more disposed to imitate than to initiate. If this is the case, the work of the Massachusetts Seismic Advisory Committee might provide a useful model for study.

CHAPTER 3: SEISMIC DESIGN DECISIONS

3.1 Committee Activities

The first meeting of the ASCE/BSCE Joint Committee on Seismic Design Criteria was held on October 1, 1973. Dr. Simpson reported on the work of the Massachusetts State Building Code Commission to prepare a uniform building code for the state to be promulgated by July 1, 1974, and to become effective from January 1, 1975. The draft code under consideration was based largely on the 1970 edition of the BOCA (Building Officials Conference of America, a Chicago based model code organization) Code. At that time the draft of the state code contained the 1970 BOCA seismic provisions. It was noted that in the area of seismic provision BOCA generally lags behind the Uniform Building Code (UBC) which is issued by the International Conference of Building Officials (ICBO) a California based code organization. The UBC generally follows the recommendations of the Seismology Committee of the Structural Engineers Association of California (SEAOC). The Seismology Committee of SEAOC had been the source of most seismic code improvements in the United States. It was noted that the 1970 BOCA earthquake provisions were not in keeping with the most recent SEAOC Seismology Committee recommendations.

This presentation introduced one of the first tasks of the Committee which was to evaluate the present seismic requirements of 1970 BOCA (and the 1972 supplement) Code in light of the 1973 SEAOC recommendations and the most recent estimates of seismic risk in Massachusetts.

Professor Whitman presented a review of the Seismic Design Decision Analysis study of seismic design considerations for highrise buildings on firm ground in Boston. He mentioned that loss estimates developed in the SDDA study suggest that expected dollar values of damage are so low that additional code requirements could probably not be justified on solely economic grounds.

The remainder of the first Committee meeting was spent in discussing the goals of Committee activity and the information needs of the Committee to reach those goals.

The goal of seismic design provisions in Californian codes was discussed. Reference was made to the SEAOC objectives of providing reasonable protection. These objectives were stated as:

For the maximum likely earthquake: no damage

For the maximum probable earthquake: no structural damage

For the maximum possible earthquake: no building collapse.

In further discussion of goals for the Committee Professor Whitman suggested the following steps:

1. Recommend to the State Building Code Commission the appropriate UBC seismic zones for Massachusetts.
2. Recommend appropriate modification of BOCA and UBC provisions for Massachusetts.
3. Recommend corrections for the apparent ambiguities in the BOCA code.
4. Write code provisions in such a way that they can be modified as information improves.

Questions raised at the first meeting centered around the issues of seismic risk, soil conditions, code reference material, and the legislative and administrative process of code development.

In the second meeting of the Committee on October 15, 1973, materials were provided on the recent research on seismic risk in New England from Mr. Richard Holt of Weston Geophysical, Inc., Mr Kaye of the U.S. Geological Survey and from the Seismic Design Decision Analysis Project at M.I.T. At this meeting the consensus in the Committee developed that it was inconsistent to design buildings in Massachusetts in accordance with requirements established for California. It was agreed that with low seismic risk the only legitimate justification for seismic design was the life safety consideration. The Committee requested further information on:

1. Ductility requirements
2. Comparison of the handling of wind and snow loads in Boston and in California
3. Comparison of Massachusetts seismicity with Zone 2 and 3 areas elsewhere.

In the third meeting of the Committee, November 1, 1973, Mr. Theodore explained that the Commission did not feel that 1970 BOCA seismic provisions were adequate but that it did not believe it appropriate to adopt the same provisions as California. He said that the Commission would probably adopt what the Committee recommended, but it did not wish to adopt requirements that would seriously escalate the cost of construction in the Commonwealth.

Professor Holley of M.I.T. provided a paper which suggested that the Commission might benefit from a statement on how to deal with the low level of seismic risk which typified Massachusetts. Prof. Holley's approach focused on avoiding hazard to life rather than risk of damage to buildings. He suggested that the code should consider the following variables:

1. geographical location within Massachusetts, as related to seismic risk;
2. soil conditions at the site;
3. function of the building;
4. density of building occupancy.

Discussion centered on the need for an independent approach to the problems presented in areas of relatively lower seismicity. It was recognized that there was a need for an approach which could balance costs and risks more effectively. A strategy had to be developed which would reduce risk at minimum cost.

During this third meeting it was decided to break the Committee down into two Task Groups: Task Group A for Soils and Seismology, and Task Group B for Structures.

3.2 Order of Study

Following the third meeting of the full Committee and the breakdown to Task Groups, most of the continuing work was carried on in the Task Groups. The seismicity studies were carried out first, then the soils work was done. Later the Structures Group was able to base its work on the conclusions of the Soils and Seismicity Groups. The Structure Group, once general policy was established on the balance of lateral force considerations and ductility considerations, basically went through UBC and SEAOC and picked the ductility requirements that would give the desired result. Some measures were included because they were cheap and others were dropped because they were considered too expensive.

3.3 Technical Decisions

The first subject to be dealt with by the Committee was the reappraisal of seismic risk in Massachusetts. The Committee had the advantage of extensive information provided by the M.I.T. Seismic Design Decision Analysis project (including SDDA Internal Study Report, "Analysis of the Seismic Risk of Firm Ground for Sites in the Central Boston Metropolitan Area" by H. Merz and C.A. Cornell, January 1972), Fr. Linehan of Weston Observatory, and Richard Holt of Weston Geophysical, Inc. Weston Geophysical was able to provide extensive material on the estimation of historic seismicity and estimates of seismic risk for New England based on studies of nuclear plant sites carried out for public utilities and the AEC. Further contributions were made by other seismologists, geologists and geotechnical engineers on the Committee.

Data on seismic risk was presented in the form of maps and graphs. Maps were presented showing location and intensity of historical earthquakes.

Other maps showed contours of intensity vs. risk for annual risk levels of 10^{-2}, 10^{-3}, 10^{-4}, and 10^{-5}. These maps were in two forms, first for intensities associated with given individual postulated source zones and then a composite map presenting maximum expected intensities considering all possible sources. Graphs were also presented of intensity vs. annual risk for the Boston area and 3 other areas of the state.

The material developed on Massachusetts seismic risk was related to expected losses in the work of the SDDA project. Material was presented on the estimation of facilities performance at various intensity levels. The SDDA damage probability matrices were presented for structures designed to various levels of the Uniform Building Code, exposed to various levels of shaking. Material was presented on the effects of soil conditions. The added hazard for buildings on soft ground was taken into account. The preliminary SDDA loss estimates for the Boston area were also presented to the Committee. It was evident from the loss estimates that because of the relatively long return periods of potentially damaging earthquakes in Massachusetts, dollar loss due to building damage would not be a major consideration of the Code. These loss estimates were based in part on detail studies of losses sustained in the San Fernando Earthquake of 1971. An attempt had been made to estimate incident or secondary costs beyond direct repair costs for damage by way of interviews with building owners in San Fernando. It is, however, still possible that the preliminary loss estimates presented to the Committee may have underestimated the broader financial implications of a catastrophic event. The economic losses due to business interruption can be enormous in the scope of state planning.

The directive to the Committee from the Code Commission was to develop measures to protect public health, safety and welfare. It seemed appropriate that questions of life safety should take precedence over economic considerations in the promulgation of a code. In this case, with the low level of expected average annual loss, the priority of life safety consideration was established.

Though it may be considered ideal that eventually a rational decision process will be able to eliminate the uncertainty and subjectivity from any technical decisions, this was not really the case in the deliberations of the Committee. The SDDA material, the seismic risk analysis, the damage probability matrices, and the loss estimates were the subject of long discussions but essentially involved the application of a combination of analysis and engineering judgment.

Eventually the Committee as a whole came to two important conclusions. First, it was generally accepted that the return period for an event giving an intensity of MMI VII+ on firm ground in Boston would be on the order of 10,000 years. This level of shaking in Boston might correspond to a hypothetical recurrence of the 1755 Cape Ann earthquake with its epicenter in Boston harbor. This order of event was accepted as a design basis earthquake for the purpose of code formulation. The decision was to design for MMI VII+ on firm ground. This decision was supported by different people for different reasons. Weston Geophysical gave this VVI VII+ on firm ground as the "maximum credible" event. Converting this intensity to mixed soft and hard ground, it was postulated that this intensity would correspond to a design acceleration of .12 g which would correspond to UBC Zone 1.5. This reasoning followed Housner's equating of Zone 3 with design for .33 g (the average peak acceleration of the El Centro earthquake), Zone 2 then became .17 g and Zone 1 became .08 g. By this logic Zone 1.5 corresponds to the geometric mean of .08 and .18 or .12 g.

The second consensus arrived at was on the question of acceptable risk. It was decided that the public probably would not accept collapse of more than 1 to 3% of structures during this design earthquake. The understanding here was that collapse was associated with loss of life. The intention was not to guarantee against any possible loss of life, but it is assumed that a 1 to 3% rate of collapse would be acceptable. (This, of course, applies only to new structures which would be subject to the Code.) The Committee, however, never voted on acceptable risk. It only made an explicit decision on an acceptable design requirement.

After agreeing upon the level and nature of the seismic risk in the state, selecting a design earthquake and an acceptable level of risk, the next step was to develop design provisions.

The Committee decided not to consider special structures in the initial version of the Code. It was decided that the Committee would limit itself to design criteria for general purpose structures because decisions on selection of particular structures, i.e. hospitals, places of public assembly, etc., would have to be the responsibility of the Commission and the legislature.

It was decided that design provisions requiring a certain minimum ductility would be more cost-effective than the establishment of a high design static force level. With exposure to hurricane winds, Massachusetts already has relatively high design requirements for wind load. Studies made available by the SDDA project indicated that a requirement for a large static seismic design coefficient would not be particularly useful, while a great need existed for establishing minimum levels of ductility to mobilize the inherent resistance of buildings. With adequate ductility, buildings can suffer considerable damage without collapse.

The Committee decided to adopt provisions along the lines of the Uniform Building Code ductility provisions for Zone 2. The design static force coefficient was set at 3/8 (this was later changed by the Commission to 1/3). These values correspond to the design acceleration of about 0.12 g. The impact of these added requirements on initial costs of construction was estimated to be at most on the order of 1% of total building cost.

From this point on, it was simply a matter of choosing from the Uniform Building Code those provisions which were considered important to ensure adequate ductility. There was remarkable unanimity once the basis for decision-making had been agreed upon. A presentation of the rationale of the proposed code was made to a meeting of structural engineers in the Boston area in Febrary, 1975, and there was little disagreement.

A probable reason for this acceptance is the fact that many engineers were relieved to see that the Code had not taken UBC Zone 3 (the level of San Francisco and Los Angeles) as had been indicated on the seismic risk map of the USGS.

3.4 Summary of Decision Points

Though the actual deliberations of the Committee as summarized in Appendix A may appear at some points inconsistent or confusing, they represent a process of probing and questioning that eventually led to the synthesis of a unique document. On examination of the recorded actions of the Committee a pattern of key decisions becomes evident.

Goals

The first concern of the Committee was to clarify its own mandate and to establish general goals for the code development activity. This involved the identification of reference points of previous practice in Massachusetts and present practice in other seismic areas (California). It also included a clarification of policy goals from the State Building Code Commission and a statement of guidelines: "acceptable" life safety at "acceptable cost".

Seismic Risk

The first object of the seismic risk study was to review the historical basis of the Algermissen map and to resolve the controversy over estimation of seismicity in Massachusetts. In this task the work of Weston Geophysical and the SDDA project at M.I.T. contributed to development of a general consensus on seismic risk. The estimation of seismic risk was used to identify a "design earthquake" and to indicate an appropriate zonation for Massachusetts.

Design Earthquake

On the basis of the seismicity studies which presented the approximate return periods for various levels of shaking to be experienced at several sites in the Commonwealth, it was possible to develop consensus in the Committee on an acceptable "design earthquake." This was considered by some to be a "maximum credible event." Others were satisfied that the estimated return period of 10,000 years provided a conservative margin of safety. The definition of the design earthquake as one giving maximum intensities of VII+ on firm ground provided the basis for selection of design criteria.

Acceptable Risk

Though not formally voted upon by the Committee, the assumption of acceptable risk for the design provisions allowed for 1 to 3% collapse of conforming structures in the "design earthquake". It was also made clear by the presentation of loss estimates from the SDDA project that purely economic incentives were not adequate to motivate the adoption of seismic provisions. The dominant issue was one of life safety. Threat to life safety was associated with particular types of building failure such as major structural failure and collapse.

Initial Costs

There was consideration of added cost implied by various levels of seismic code regulation. This information was supplied by SDDA and was limited to several types of frame buildings. The Committee had a feeling for the costs of various levels of protection.

Added cost to structural frame (over Zone 0):

Zone 1 - 0

Zone 2 - 5%

Attention centered on the comparison of implications of Zone 1 and Zone 2 and on the possibility of a compromise. The major issue in this consideration was the relative importance of lateral force requirements and ductility requirements. Zone 2 forces without ductility requirements were considered and dismissed. It was recognized that inclusion of ductility requirements was essential to reduction of seismic risk in Massachusetts.

Lateral Force and Ductility Requirements

There was not a clear understanding of the relationship of damage estimates to potential life loss. It was assumed that most fatalities were associated with total collapse, so the logical goal of the code was to avoid collapse in the design event. In keeping with the goal of emphasizing life safety rather than property protection, greater emphasis was put on provision to prevent collapse. The requirements of the UBC were reordered to distinguish ductility requirements from lateral force requirements. Because Massachusetts buildings are designed to resist wind loads, many already have significant lateral force resistance. However, even in the earlier Boston Code and School House Code which recognized earthquake risk, no ductility requirements had been imposed.

SDDA studies had shown high static lateral force requirements to be less effective in increasing life safety than ductility requirements. It was also felt in the Committee that ductility requirements would be cost effective; that is,at a minimal added initial cost a significant reduction in collapse and life safety threat could be achieved. The Massachusetts seismic provisions were designed to emphasize means of acheiving adequate ductility.

Zonation

On the basis of the seismic risk studies it was possible to identify UBC 1.5 as the appropriate zonation for Massachusetts. The issue of variations in seismic risk across the state was subordinated to the interest in developing a uniform statewide code. It was also indicated in the seismicity studies that the western part of the state may be affected by earthquake originating in source zones other than those affecting the eastern part of the state. (It should be noted that on a "Preliminary Map of Horizontal Acceleration (Expressed as Percent of Gravity) in Rock with 90 Percent Probability of not being Exceeded in 50 Years" (U.S. Geological Survey Open-file Report 76-416, 1976), the levels of acceleration are shown to be more uniform across Massachusetts than would have been suggested by the 1969 Algermissen Map.)

Soil Factor

It was decided by the Committee to include a soil factor as a multiplicative factor in the equation for base shear. (The 1973 UBC on which the Committee's work was partially based did not yet include a soil factor.) The Committee decided that for the sake of simplicity it was reasonable to consider only two categories of soil, soft and firm. For some parts of Boston, with 120 feet of clay, the soil factor is 1.5 which means that the seismic coefficient approaches 1/2 rather than 1/3 as in the case of firm soil. (This is the reason for the adjustment of the Z factor to 1/3 rather than 3/8. The Commission changed the Z factor to 1/3 in order that S[1.5(soft soil factor)] x Z[1/3] = 1/2. 1/2 is the Z factor which had previously been required in Boston under the Zone 2 UBC requirements of the 1971 Boston City Code.)

3.5 Inputs of SDDA to the Development of the Massachusetts Seismic Provisions

Excerpts from the SDDA project were presented at various times and information generated by the project was provided through the participation and contribution of project staff members in the work of the Committee.

The most important inputs of SDDA were:

1. Seismic risk analysis:

   Graphs and maps on seismicity in New England including risk curves of return period vs. intensity for various sites in Massachusetts.

2. Loss estimates:

   Summaries of expected building performance for various levels of design at various intensities of shaking. These included damage probability matrices which related damage to experienced intensities on firm and soft soil.

3. Initial cost estimates:

   Summaries of a pilot study of the added initial building costs associated with various levels of UBC design requirements provided valuable input for assessing the expected immediate economic impact of seismic design provisions.

These materials along with studies comparing events which might be expected in Massachusetts with experience of events causing similar intensities elsewhere (especially Caracas, 1967) were the subject of intense discussion and offered guidance in the crucial decisions on seismic risk, design ground motion, acceptable risk, ductility provisions, zonation and soil factor.

Much of the SDDA input came directly to Task Group A (soils and seismology) which was chaired by Professor Whitman.

CHAPTER 4: THE FINAL PRODUCT: IMPLEMENTATION AND RESPONSE

4.1 Adoption of the Seismic Provisions

In January 1975 the Commonwealth of Massachusetts State Building Code came into effect. This is a mandatory statewide building code which is required of every town and municipality in the state. It is the only code allowed and no modification is allowed without the approval of the State Building Code Commission. The Code is largely based on 1970 BOCA and the 1972 BOCA Supplement. Though it takes the same general approach and philosophy, there are some significant differences. One of these areas of difference is the Section 718.0 Earthquake Load.

The recommendations of the Seismic Advisory Committee were forwarded to the State Building Code Commission in December of 1974. The recommendations reflected the results of studies of the probable intensity and frequencies of occurrence of future earthquakes in the Commonwealth and represented the Committee's opinion as to the minimum appropriate standards of design and construction for reasonable protection against structural collapse due to a major seismic event. The letter of transmittal which accompanied the Committee's recommendations stressed that the provisions could hope to achieve their goal only if construction is carefully inspected by a knowledgeable, competent inspector guided by instructions or inspection specifications prepared by the design engineer.

The draft of the seismic provisions was adopted, except for minor editorial and regulatory language changes as proposed, with two exceptions.

The first exception was to alter the desired requirements that the Committee presented for inspection of construction. These requirements were relegated to language that placed them within the province of rules and regulations to be promulgated by the State Building Code Commission. This was done for three reasons: first, it was felt that part of the problem was adequately addressed by the state's own very special provisions for control of construction. Although this was not completely adequate, it did provide for extensive control of inspection in many cases. Second, the state code is mandatory and uniformly applied and with all of the other problems of transition involved in the changeover to the new code, the inspection as proposed was more than could be incorporated into the regulatory system at the time. Third, the state was attempting to establish general requirements for licensing of individuals and firms involved in the testing and inspection in all areas required by the building regulatory system. This program is not yet developed to the point where it is possible to establish general requirements for special inspection of buildings and structures designed subject to the earthquake provisions.

The second exception is the lack of specialized provisions for masonry, and this exception developed more by default than by intent. The Commission never intended to delete any special requirements for masonry. However, the Commission did recognize that in certain small structures and one- and two-family dwellings special prescriptive requirements would have to be outlined to present as simple a design procedure as possible. By default of time on the part of the staff, this was never done before promulgation and the staff is still in default. The deletion of the masonry provisions has also been described as an action taken in the hope that a "more meaningful" section could be developed which would relate to anticipated developing of licensing procedures for building supervisors. The inspection indicated by the Committee for masonry was by the Commission deemed to be unfeasible at the time due to the limited training and experience of inspectors.

In February of 1975 the Seismic Advisory Committee arranged two workshops at the Boston Public Library for design professionals, engineers and architects, to explain the content and intentions of the new seismic provisions. The presentation was made by Dr. Simpson, Dr. Zaldastani and Professor Whitman.

There was widespread professional interest in the recommendations of the Seismic Advisory Committee. This interest was evidenced by the size of the audience at the Committee workshops. The engineering profession took an active honest interest in protecting the area from unreasonably restrictive regulations. Previously there had been a real problem of inequity. The Algermissen map had put Boston in Zone 3 and according to the previous code this would have implied prohibitively expensive construction.

Previous to the presentations and discussions at the Boston Public Library there had not been any formal or large scale consultation with the local engineering community outside the Committee. However, the combined result, Section 718.0 of the new State Code, was apparently acceptable to professionals and the public. The Joint ASCE/BSCE Committee was generally regarded as a blue ribbon committee and had adequate prestige that its conclusions met general acceptance and had the broad support of the engineering community.

Generally speaking, codes are apparently not seriously considered by industry or the design professions until they are applied in practice. The fact that there has been no major dissatisfaction may be due to the low added initial cost implied by the seismic regulations. It should also be noted that in the period since the promulgation of the State Building Code the building industry in Massachusetts has been severely depressed. This may mean that the true impact of the seismic provisions on design and construction practice and costs is still to be felt.

4.2 Industry Response

Several building material industry groups requested the opportunity to address the Committee. There was no resolute opposition to change. The masonry group did not object to requirements for reinforcement of all masonry construction. Some detail provisions on the minimum spacing of bars were questioned but in general there was acceptance. The steel industry also had some comments of a practical nature. Some misunderstanding which had developed on the basis of obsolete data and misinformation were cleared up in subsequent discussions. Though there had been fear of a solid resistance from industry to many of the recommendations, this resistance did not materialize.

In general the comments of representatives of building material groups were helpful. Two examples of interest are the contributions of the Portland Cement Association and the American Iron and Steel Institute.

Mr. George Brattin of the Portland Cement Association provided the Committee with a summary of Ductility Provisions of the Body of the Building Code Requirement for Reinforced Concrete (ACI 318-71). In this summary Mr. Brattin argued that the ACI Code was a ductile code and that the provisions of the main body of the ACI Reference Standard 318-71 were adequate for New England and that the provisions of ACI 318-71 Appendix A (which UBC requires for Zone 2) or any other additional ductility requirements are unnecessary in Massachusetts. This Summary was of course very helpful as it pointed out the specific ductility requirements in the body of the ACI Reference Standard. However, it also has to be recognized as a defence of reinforced concrete construction. The imposition of strict ductility requirements would clearly have some negative effect on the competitiveness of concrete structures compared with, for example, steel.

Mr. Daniel M. McGee of the Committee on Construction Codes and Standards, American Iron and Steel Institute provided comments which questioned the Massachusetts seismic provisions as not being adequately stringent. Mr. McGee questioned the justification of the Massachusetts seismic provisions in departing from the examples of the SEAOC recommendations and the Uniform Building Code. The AISI comments included a recommendation that the Z value for eastern Massachusetts should be 3/4 and the rest of the state 1/2 as opposed to the uniform value of 1/3 found in the Massachusetts provisions. There was also an argument for a less conservative treatment of steel on the basis of its past performance. AISI felt that the treatment of steel was inconsistent with the treatment of other materials like reinforced and prestressed concrete, which have not performed as well as steel in the past. The overall intent of the AISI comments may have been somewhat self-serving since more stringent seismic requirements would tend to put concrete at a disadvantage as compared to steel. Detail comments were, however, helpful.

Generally, in their first meetings with the Committee, the representatives of materials groups made confident and aggressive presentations of their points of view. However, when they realized the considerable expertise of the Committee their participation was characterized by a spirit of cooperation rather than confrontation. The meetings of the Committee with industry representatives were enlightening experiences for both parties. In the case of the Prestressed Concrete and Masonry industry, representatives were content to claim that prestressed concrete and masonry could resist the lateral force requirements of the Code. The response of the Committee was to rephrase the problem of meeting the intention of the Code by asking what provisions must be adopted to ensure safety during the 0.12 g earthquake. Such questions forced the materials industry people to think in terms of ductility also rather than simply resistance of lateral forces. The description of the design requirements by a 0.12 g earthquake and response spectra, with the privilege of using smaller forces in design IF ductility requirements were met, was an important and useful innovation.

The response of the building material industry to the Code was not an expression of opposition to the spirit or content of the seismic provisions, but rather a concern for the impact of the seismic provisions on the relative competitive status of the various materials groups.

4.3 Level of Compliance with the Code

There is a possibility that the lack of greater controversy or opposition to the seismic provision is an indication that they are being ignored in those areas where they present problems for designers or builders. To date there have not been any objective studies of the actual impact on design practice or construction costs. There is no certain way to estimate what number of design offices responsible for structures in Massachusetts were involved with seismic design in any detail prior to the promulgation of the Code. The suspicion is that in many towns in the Commonwealth, for most of the moderate and small sized buildings, there was token appreciation of requirements for windloads and probably none for seismic.

However, the effect of going from what was either a non-existent or minimum code throughout most of Massachusetts to a relatively comprehensive code produced very little apparent confusion or upset.

The seismic code is more dependent upon design professionals than building officials. The effectiveness of a seismic code depends on more qualified inspection and enforcement. It is the designers (engineers and architects) and the building contractors who have to apply the seismic code. The seismic provisions are an example of well-applied expert advice. They stand as a relatively isolated section of the Code. Though they are very complex in origin, they are presented clearly and unambiguously. The seismic provisions were written directly by people with relevant technical expertise. They are not as detailed as the seismic section of the California code. Few people ask for clarification.

Though there may be some instances of engineers misapplying or bending the rules of the new seismic provisions, this will be less and less the case in the future. Engineers are more and more being held liable for shortcomings of their designs. Any collapse or loss of life resulting from failure to comply with code regulations will place the engineer in serious jeopardy. It is quite clear to the profession that anyone foolish enough to ignore his professional responsibilities by ignoring any part of the Code is courting disaster.

For some professionals the imposition of seismic design regulations by the State Code will come as a relief as it will serve to establish a uniform level of safety. In the case where such considerations are voluntary, the competitive market makes it difficult to motivate increases in design cost on an individual basis.

Professional Orientation

For the engineering community, the transition to the new seismic provisions was greatly aided by a series of lectures sponsored by the Boston Society of Civil Engineers Section, American Society of Civil Engineers eight months after the provisions came into effect. The series of seven lectures was sponsored in cooperation with the Massachusetts Institute of Technology, Department of Civil Engineering. The series included lectures on:

1. Introduction, Fundamentals, Description, Massachusetts Building Code Provisions.
2. Designing Reinforced Concrete Buildings for Resistance to Seismic Effects under the new Massachusetts State Building Code.
3. Dynamic Analysis of Buildings for Earthquake Design.
4. Soils and Foundations.
5. Structural Steel.
6. Masonry.
7. Prefabricated Construction Systems.

Up to the time of the lecture series, several technical questions per week of varying complexity on the provisions were addressed to the Commission. Subsequent to the lectures the number of inquiries was reduced to somewhat less than one per week. There has not been any overt concern about the additional financial burden generated by the provisions.

The development of the state seismic provisions fulfilled a very real and special need for the state. The results appear to be very successful. It should be noted, of course, that there was a unique and remarkable concentration of talent available to the Commission to accomplish the work. The Committee and the Commission anticipate future changes in the provisions. However, the major change from a situation of virtually no seismic regulation to the present one in which designers must take into account seismic consideration has been successful. The change has caused minimal confusion, and there has not been extensive negative reaction concerning the economic impact of the seismic provisions.

CHAPTER 5: THE FUTURE OUTLOOK

5.1 The Status of the Committee

The Seismic Advisory Committee has now been made a member of the Code Technical Council to advise the State Building Code Commission on allowance of variances, and updating and revision of the seismic provisions. The Committee will monitor progress in the area of seismic design codes and propose revisions from time to time as they become appropriate. The Committee is still constituted on an unpaid, voluntary basis.

Since the Committee formulated its recommendations to the Commission, further issues have been recognized for consideration and significant developments have taken place in the field of seismic design regulation.

5.2 Unfinished Business

It is recognized that as promulgated and enforced, the Massachusetts State Building Code seismic provisions suffer from some significant omissions. These omissions are in the coverage of provisions for masonry construction, occupancy importance classification, existing buildings and enforcement of the provisions. These problem areas need to be dealt with in order to complete the original task of the Committee independent of subsequent developments in research and regulatory practice elsewhere.

Masonry

At present the code requirements for ductility in masonry as given in 718.53 MASONRY, are unclear and subject to misinterpretation. Section 718.53 simply states that masonry shall be subject to the provisions and reference standards of Article 8. Article 8 does not require reinforced masonry for seismic resistance. Reference standards given in Article 8 cover requirements for both reinforced and unreinforced masonry. However, there is no indication of when reinforced masonry is required. A requirement for reinforced masonry may be indirectly inferred from Section 718.56, Other Materials or Methods of Construction, which does not specifically approve unreinforced masonry as a structural system. However, the vague and imprecise nature of the treatment of masonry has in fact resulted in no change in masonry construction practice to increase ductility. Section 718.56 does apparently require that unreinforced masonry structures should be shown to safely withstand lateral distortion eight times that computed for the lateral forces specified in Section 718.4.

The recommendation of the Seismic Advisory Committee for Section 718.53 is as follows.

Masonry

a) All bearing walls, shear walls, exterior walls, chimneys and parapets, which are constructed of masonry shall be reinforced in two directions so as to qualify as Reinforced Masonry according to the provisions of the NCMA Specification for the Design and Construction of Load Bearing Concrete Masonry or the BIA Structural Clay Products Institute Recommended Building Code Requirements for Engineered Brick Masonry.

   In masonry bearing or shear walls principal reinforcement shall be spaced a maximum of 2 feet on center in either the horizontal or vertical direction. In the other direction spacing of reinforcement may be increased to 4 feet.

   Non-structural masonry walls which enclose stairwells or elevator shafts, other than exterior walls, shall be designed as Partially Reinforced Masonry in accordance with the Standards listed in the Article 8, Part B. Spacing of reinforcement is not to exceed 4 feet.

b) Columns

   The size and spacing of ties at the ends of tied columns shall not be less than those required for concrete columns. See 718.51 (b).

c) Anchorage

Masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal design forces or a minimum force of 200 pounds per lineal foot of wall, whichever is greater. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall.

This clarification of requirement for ductility in masonry construction is essential to fulfillment of the basic philosophy of the Code. It is also of utmost importance in view of the demonstrated vulnerability of unreinforced masonry structures in past earthquakes and the prevalence of masonry construction in Massachusetts.

Occupancy

The inclusion of occupancy classifications for an importance factor were considered by the Committee during the formulation of the original recommendations. Such an importance factor for design of critical facilities had been proposed by SEAOC in 1973. The Committee decided that the ranking of socially important facilities was a political question, not a technical one. As a result there is no importance factor in the Massachusetts Code. However, the 1976 Edition of the Uniform Building Code does include an importance factor for essential facilities. Essential facilities are defined as those structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public. Such facilities include:

1. Hospitals and other medical facilities having surgery or emergency treatment areas.
2. Fire and police stations.
3. Municipal government disaster operation and communication centers deemed to be vital in emergencies.

The design and detailing of equipment which must remain in place and be functional following an earthquake can justifiably be required to meet more stringent requirements than other types of facilities.

The judgment of the Committee that the ranking of socially important facilities constituted a political judgment is of interest as it indicates a recognition on the part of the Committee of some boundary between political and technical issues in code formulation. It may be questioned to what extent the implicit decisions the Committee made on acceptable risk were "technical" rather than "political".

Existing Buildings

Questions have arisen as to how to deal with the hazard presented by existing pre-code buildings. A general assumption has been that aside from special purpose structures, such as hospitals, no retroactive requirements can be enforced, except in the case of major renovation.

The case of a recent addition to a hospital in Boston serves as an example of the existing building problem. The new addition was adjacent to an existing masonry building which probably would suffer severe damage in a major earthquake. Recommendations were made to strengthen the building. The trustees of the hospital proposed that if the building's projected lifetime were limited to 10 or 20 years, this would have the same effect on the level of risk as strengthening the building. This may be a reasonable way to deal with earthquake risk. The probability of experiencing a major earthquake in the next 20 years is certainly less than for the next 100 years, the projected life of a new building. This reasoning has been applied to dealing with the earthquake hazard presented by existing pre-code buildings in the City of Long Beach, California (Wiggins, J.H., and D.F. Moran, Earthquake Safety in the City of Long Beach Based on the Concept of Balanced Risk, September 1971, J.H. Wiggins Co.)

There will have to be further consideration of the evaluation of seismic risk for existing and temporary structures. Studies will have to be directed toward the feasibility of reducing seismic design requirements in accordance with expected building life.

The problem of seismic risk in existing buildings is particularly acute in the case of recycling and renovation. Renovation constitutes an extension of the expected life of a given building and therefore affects the likelihood of its experiencing a damaging earthquake. A rational basis for establishing acceptable levels of risk may be based on the principle that risk to an occupant in any building at any given time should be equal. Any deviation from this principle should require justification. In the case of renovation in Massachusetts there are definite cultural values and economic conditions which might argue for variance in the level of acceptable risk, but at present these relationships are very poorly defined. The general response in this area of uncertainty has been to grant blanket variances for renovation of historic and pre-code masonry structures undergoing restoration. Such action may in fact preserve major hazards to public safety and stand in direct contradiction of the philosophy and objective of the Code.

Beyond the issue of whether or not to consider seismic risk in existing buildings is the problem of how to evaluate existing buildings. There is a general presumption of a correlation between the force function and risk of failure for existing buildings. This may not be the case. The main problem with pre-code structures is that they demonstrate no consideration of earthquake shaking. The difference between no consideration and some consideration is of much greater importance than some detailed adjustment of the actual lateral force function in design.

The lateral function provides a convenient variable for political manipulation, but it is of little value in assessing the earthquake resistance of existing buildings. Checking the resistance of an existing building with the horizontal forcing function is not a valid means of establishing the probability of experiencing earthquake damage. Many of the causes of earthquake damage have to do with the attitude of the designer. To verify an existing building for .1 g or .2 g tells one little about the probability of survival of a .1 g level of shaking. Caution must be exercised in applying analytical methods which are appropriate for new construction to existing buildings.

In a period in which expenditure on renovation has exceeded total expenditure on new construction it is particularly important that adequate provisions be developed to achieve an acceptable level of seismic risk for existing, pre-code buildings in the cases of occupancy change or extension through renovation.

Non-Structural Elements

A further concern for future consideration is the addition of seismic design provision for critical non-structural building elements. At the levels of shaking considered probable for Massachusetts, there is likely to be considerably more damage due to non-structural failures than to major structural collapse. The types of failures which are likely to occur at relatively low levels of shaking include the collapse of parapet walls, the loss of cladding and the disruption of mechanical systems. Loss of life is likely to result from the failure of non-structural building elements. Subsequent additions to the seismic provisions should include consideration of critical non-structural elements, especially in critical facilities which will be needed in the aftermath of an earthquake.

Inspection

The new code does not include expanded state level enforcement. The local level is still responsible for enforcement. Now that there is only one code, the state inspectors have become redundant. In order to retain their jobs they are now considered Advisors to the local inspectors. The local inspectors are now responsible for all buildings in their area. The local building official is now required to be consistent with the state code.

The recommended seismic provisions for masonry were deleted in part because a lack of adequately trained and experienced inspectors to enforce them.

The seismic provisions represent a significant increase in the level of complexity of the Building Code. They assume a level of understanding on the part of the building official which is not always the fact. The seismic provisions also imply greater costs in terms of site inspection. If the inspection and enforcement system are not given adequate support and training to apply the Code, its impact in reducing threats to life safety will be minimal.

As the level of sophistication of the Code increases it becomes imperative that building officials be accorded professional status. The political nature of building official appointment has hindered the development of an adequate level of professionalism. The complexity of the present four tier system of inspection and enforcement in Massachusetts also serves to frustrate both the efforts of building officials and the full implementation of the Building Code.

5.2 Response to Future Inputs

In monitoring future developments in seismology, earthquake engineering, and building regulation, the Seismic Advisory Committee will in effect be reviewing its earlier decisions in light of new evidence. A particular case of such new evidence is the publication of the Recommended Design Criteria of the ATC-3 program of the Applied Technology Council. Other future inputs which may be reflected in the seismic provisions are improvements in the estimation of regional seismicity and advances in the field of earthquake prediction.

ATC-3

The Applied Technology Council, with the support of the National Bureau of Standards and the National Science Foundation, is completing the development of comprehensive, nationally applicable seismic building code provisions with the expectation that these new provisions will replace the existing seismic design provisions of the model codes. The new seismic design criteria are intended to incorporate the present state-of-the-art. It is of interest to make a preliminary comparison of the requirements which would apply in Massachusetts according to the ATC-3 recommendations and those of the present Massachusetts Code (for Zone 1.5 or Z = 1/3).

- For ductile concrete frame buildings, the lateral force requirements of ATC-3 are twice those of the Massachusetts Code
- For regular concrete frame buildings, the lateral force requirements of ATC-3 are three times those of the Massachusetts Code (assuming reinforced masonry infill).
- For shear wall buildings, though the factor varies according to actual plane geometry, ATC-3 indicated a 20 to 30% increase in lateral force requirements over the Massachusetts Code.

In order for the implications of ATC-3 for the Massachusetts Code to be judged, there will have to be some consideration of the way in which ATC-3 has dealt with the nature of the seismic problem. It is possible that ATC-3 reflects aspects of California's earthquake experience which do not apply in Massachusetts. It is quite probable that there are differences in expected duration of shaking. Also it would be necessary to consider the underlying assumptions regarding levels of acceptable risk.

In any event, if the ATC-3 recommendations are sanctioned by the federal government and the model code groups there will be heavy pressure on the Seismic Advisory Committee to conform or at least to clarify the basis for deviation from the nationally recommended criteria (See Appendix C).

Improved Estimates of Regional Seismicity

There has been some controversy over the fact that the Massachusetts Building Code has uniform seismic provisions for the entire state while in the seismic risk map prepared by Algermissen only Boston and the east coast are in Zone 3 while most of the state is in Zone 2, with the southwest corner in Zone 1.

The Algermissen map is based only on maximum experienced events and does not take into account the detailed estimate of seismic risk due to all sources that was made available to the Committee. Also, it was felt that the error band on the determination of required ductility and seismic force coefficient was such that it was not meaningful to distinguish between the various projected intensity levels over the state. While the eastern part of the state may be subject to an earthquake originating near Cape Ann, the western part may be subjected to one originating in the area of the St. Lawrence or in New York State. Though there is a difference in the probability of these events, it did not seem great enough to justify distinguishing different code provisions for different parts of the state. However, it is possible that in the future, if there is discontent in the western part of the state, that changes may be made to reflect the relatively lower seismic risk there. Such a change may be based either on a reevaluation of the trade-off between the advantages of uniformity in the code and building cost in the western part of the state or because a better basis for discrimination of seismic zones becomes available. Similarly it is possible that the reevaluation of historical records or new information of other origin may lead to a change in the description of the design ground motion.

Prediction

The design earthquake is estimated to have an annual probability of occurrence of roughly 10^{-4}. In general, damaging earthquakes in the Northeast have fairly long return periods. Because of the low probability of damage, it is difficult to justify building reinforcement on strictly economic grounds. If a reliable means of earthquake prediction is developed which could provide time for selective strengthening or evacuation with earthquake prediction, the threat to life safety could be dealt with with reduced reliance on seismic reinforcement. Seismic provisions might then only be necessary for those buildings which house functions of critical importance during and immediately after an earthquake.

Because we are primarily concerned with life safety and not economic loss, we need more information on how to prevent complete collapse and what is really needed to mobilize to the maximum practicable extent the available seismic resistance of a structure. In view of the long return period of damaging earthquakes in Massachusetts, we may have to approach the problem of seismic code provisions differently from California.

The advent of effective earthquake prediction would considerably change the handling of economic and social factors in the code. Once a potential target area becomes known its effective seismic risk rises dramatically. Thus using the same methodology very different levels of investment to avoid damage can be justified.

SUMMARY AND CONCLUSIONS

The activity of code writing is very time-consuming. It is difficult to assemble the talent and time of the profession together to issue guideline documents. In the two years spent by the Committee it is estimated that something on the order of 1500 man-hours were spent in meetings alone. This does not include the time and cost of individual preparation. From this considerable collective effort came a document which represented both a critical review of existing data and precedent and a number of significant innovations. The work of the Seismic Advisory Committee to the Massachusetts State Building Code Commission represents the first formal effort to develop seismic design criteria specifically for an eastern seismic area. For this reason, it also signifies the first recognition of some important differences between strategies developed to cope with eastern seismicity and those developed to cope with the relatively higher level of seismicity in the West.

Among the major contributions of the Committee the following should be listed:

1. The rational selection of a design earthquake based on detailed seismic risk analysis.

2. The introduction of the concept of a design ground motion and description of the design requirements in terms of a 0.12 g earthquake with response spectra. The allowance of an option to design for smaller forces IF ductility requirements are met.

3. The reordering of seismic design provisions to emphasize the distinction between lateral forces and ductility requirements.

4. The recognition that inclusion of more stringent ductility requirements may be of greater value in reducing seismic risk in the East than increasing lateral force requirements. This is expressed as an allowance for design to smaller lateral forces IF ductility requirements are met.

5. Improved presentation of lateral force requirements.

6. Selection of applicable ductility requirements for use in Massachusetts.

7. The development of soil factor provisions.

8. The development of liquefaction provisions.

The Role of SDDA

Beyond the value of the final recommendations of the Committee, there is also something to be learned from the way the Committee went about its work. With the support and reference of the Seismic Design Decision Analysis Project the Committee was able to deal in a systematic way with the questions of seismic risk, damage probability, initial costs, loss estimate and in a very rough way to balance or optimize the balance of initial costs for earthquake resistance with the costs of expected losses. The SDDA estimates of seismic risk were essential in the selection of the design ground motion. The SDDA soil studies were critical in the development of the response spectra and the soil factor. The SDDA damage studies played a major role in the selection of design provisions and the emphasis on ductility. The SDDA loss estimate made clear the relative proportion of the life safety and property protection issues in Massachusetts. The SDDA estimates of initial cost impacts were critical in the determination of an economically acceptable level of regulations.

The work of the Committee followed a fundamentally rational order. Supporting information was provided to the extent possible for every decision point along the way. The Committee began by clarifying its specific goals and then systematically dealt with each in order. In general the Committee was aware of the complex meshing of technical and social issues involved in any life safety code. In some cases the Committee felt little constraint in making what might be considered political or social decisions rather than technical decision. In other instances the Committee showed more than scrupulous restraint in dispatching "political" questions.

Influence of Point of View

When deriving values for code requirements, the tendency is to look at averages. The insurance actuary looks at averages. A large organization with many installations can look at averages. Large organizations can respond to figures like expected average annual loss. However, when interest is limited to one particular structure, average expectations no longer have significance. It is no consolation for the owner of a collapsed structure to know that his structure is balanced by others which are stronger than necessary. There is still a need for a good analytical technique which can be applied to specific cases.

Those designing according to the Code must understand that the Code provisions are based on averages. The seismic provisions of the Massachusetts Code represent the selection of a design level such that an earthquake generating an intensity of VII.5 on firm ground would result in only about 1 to 3% collapse. This rate of collapse represents an average for a population of a number of classes of structures of various materials. If an owner constructing an individual building wants positive protection against collapse in an earthquake of that intensity, he should not be led to believe that simply designing by the Code is adequate. The Code represents a minimum standard for an assumed level of risk for a large population.

There are tools for checking the resistance of individual structures. Building owners must understand that the Code represents a maximum acceptable average risk from the public point of view, and that fulfillment of Code requirements does not automatically provide the margin of safety which the owner may want. Important structures must, of course, be given considerably more attention than is implied by simply meeting the Code.

The Massachusetts Seismic Advisory Committee as an Example for Other Seismic Areas in the East

The Boston area is probably not typical in terms of the availability of technical expertise. Because of the presence of major institutions of higher learning and research, and engineering firms with international practices, there has been a deep and broad source of professionals with experience in seismology and seismic design. This is certainly not the case in many smaller communities in the country, which are now faced with the question of dealing with seismic risk. The type of effort initiated and carried out by the Seismic Advisory Committee would not be possible on a consultant basis in a small community. (It is estimated that the 1970 revision of the Boston City Code cost on the order of $100,000 and that the recent revision of the New York City Building Code cost $800,000.)

The process of transferring technology into policy involves presenting data and technical results in a simplified form which can be understood by decision-makers. Basic parameters should be clear to allow for well-structured choices at the local level which would reflect local conditions and local judgments on acceptable risk. The consequences of various Code provisions must be evaluated in terms of effectiveness in risk reduction and in terms of added construction cost. A major technical problem is estimating the impact of design requirements. A rational decision methodology should break down the code development process to a series of basic paramaters. It should be made clear where the critical decisions lie and what input they should be based on. The code development process could be systematised to the point that a competent local group in any community could arrive at decisions which represent their views on the appropriate balance of risk and cost.

Reference people with special expertise in the field of seismic design decision-making will have to be identified and provided on a consultant basis to advise local code groups. Their major role will be to indicate what decisions need to be made and to provide, or at least give guidance as to how to obtain relevant facts for decision-making.

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A

APPENDIX A

CHRONOLOGY OF ACTIONS IN THE DEVELOPMENT OF THE SEISMIC PROVISIONS OF THE MASSACHUSETTS STATE BUILDING CODE

I. Background

1947

State Building Code promulgated by State Board of Standards (non-mandatory, based on BOCA).

1950s and 60s

Atomic Energy Commission studies of seismic risk in New England.

1962

1962 Edition of the Building Code of the City of Boston. (No mention of earthquake consideration.)

1968

Legislation passed creating the Massachusetts Department of Community Affairs. The Governor and General Court direct the Department to submit a Model Building Code by December 4, 1970.

1969

U.S. Geological Survey permits publication of a revised seismic risk map of the United States prepared by S.T. Algermissen. The Algermissen map places Boston and the Northshore in Zone 3, most of the state in Zone 2 and the southwest corner in Zone 1.

July 1, 1970

Revised Building Code, City of Boston, published by Building Department of Boston. Result of consultant effort carried out by Francis S. Harvey and Herbert Eisenburg. Accepts Zone 2 for Boston and by reference UBC 1967, Vol. 1, Section 2314, "Earthquake Regulations." Requires only "capable of withstanding lateral forces;" remains vague on the question of required ductility.

December 4, 1970

Report Relative to the Development, Administration and Enforcement of Building Codes submitted by the Department of Community Affairs to the Governor and the General Court. Recommendations were made for a mandatory statewide building code, based on performance criteria, to be uniformly administered, interpreted and enforced.

January, 1971

Funding of NSF sponsored research project in the Civil Engineering Department at M.I.T. on Optimum Seismic Protection for New Building Construction in Eastern Metropolitan Areas. (Research activity came fully under way by September 1971.)

July 1, 1971

Boston City Council voted to adopt the Boston Building Code as published in July 1970.

1972

Passage by the Massachusetts General Court of St. 1972, C. 802, the enabling legislation for the promulgation of the State Building Code and the establishment of the building regulatory system. This was done with the full bipartisan support of Governor Francis W. Sargent, Senate President Kevin B. Harrington, Speaker of the House David M. Bartley, and members of the General Court, as well as that of Richard E. McLaughlin, Secretary of the Executive Office of Public Safety.

1972

Presentation of the S.T. Algermissen map for a meeting of structural engineers in Boston. (Seismic Risk Map of the Conterminous United States from S.T. Algermissen, "Seismic Risk Studies in the United States," Proceedings of the Fourth World Conference on Earthquake Engineering (Vol. 1 pp. 19-27) Santiago, Chile, 1969)

February 21 and March 14, 1973

Presentation of preliminary results of the Seismic Design Decision Analysis project at M.I.T. to the Boston Society of Civil Engineers by Professors Whitman, Christian, Cornell, Vanmarcke and Mr. Brennan (See SDDA Report 10). Attended by 150 engineers from the Boston Area. See: Whitman, R.V., J. M. Biggs, J. Brennan II, C.A. Cornell, R. de Neufville, E.H. Vanmarcke, Methodology and Pilot Application, Seismic Design Decision Analysis Report 10. Department of Civil Engineering, M.I.T. MIT-CE-R74-15 Structures Publication No. 385, Cambridge Mass. July 1974.

II. Formation of ASCE/BSCE Joint Committee

May 2, 1973

Letters sent by Professor Whitman to Mr. Max Sorota, President of the Boston Society of Civil Engineers and to Mr. Ronald Hirshfeld, President of the American Society of Civil Engineers, Massachusetts Section, suggesting the formation of an ASCE/BSCE committee to review the seismic design requirements currently in effect in the Commonwealth of Massachusetts and, if appropriate, to recommend changes.

June 29, 1973

Howard Simpson agreed to serve as chairman of an ASCE/BSCE Joint Committee on Seismic Design Criteria. The Joint Committee on Seismic Design Criteria membership is chosen to represent expertise in structures, soils, and seismology. Provision is made for working and liaison members from major engineering firms and state agencies.

July 27, 1973

News item on the Seismic Design Criteria Committee published in the ASCE Forum describing committee activity and membership. Committee officially established to provide input to the State Building Code Commission on the question of seismic design requirements prior to January 1, 1974.

September 1, 1973

A three year grant was awarded by National Science Foundation to Professor R.V. Whitman for continuation of M.I.T. research entitled: "Seismic Design Decision Analysis for Eastern Metropolitan Areas."

III. Decision Process

October 1, 1973

First meeting of the Joint Committee on Seismic Design Criteria. Simpson reported on the present work of the State Code Commission to prepare a uniform state code by January 1, 1974. The state code is essentially a modified BOCA. The state code will constitute a required minimum. A bill now under consideration in the legislature would replace all city and local codes with the uniform state code by January 1, 1975. At present the draft of the state code contains BOCA earthquake provisions. BOCA is a Chicago-based model code organization which generally lags behind ICBO, a California-based model code organization, in earthquake questions. BOCA follows UBC (ICBO) and UBC genereally follows the recommendations of SEAOC which is the Structural Engineers Association of California. The seismology committee of SEAOC has been the source of most seismic code improvements in the United States. The BOCA Earthquake Provisions of 1970 (Fifth Edition) are very general and ambiguous. The recommended earthquake lateral forces of the code are not in keeping with most recent estimations by SEAOC and there is no mention of ductility requirements.

Further information was provided concerning the composition of the Building Code Commission. The State Building Code Commission as constituted includes representatives of a range of involved groups, labor leaders, architects, engineers, materials suppliers, building owners, and the fire marshall.

In 1973 when code development work was in progress, Donald Stull was chairman of the Commission and Charles Theodore was the only engineer member. The Commission held hearings in each county to accept testimony on 1970 BOCA Code and the 1972 cumulative supplement. Mr. Theodore was responsible for any revision of the earthquake provisions and welcomed the support of the ASCE/BSCE Joint Committee on Seismic Design Criteria.

Professor Whitman reported that the National Bureau of Standards is in the process of developing National Recommended Seismic Design Criteria and that it would be advantageous for Massachusetts to be actively involved rather than to find itself in the situation of reacting to an accomplished fact.

Discussion of Goals (for the following 3 months):

1. Consideration of seismic risk.
2. As time was too limited for microzone study of the state, it was suggested to take representative studies for guidelines.
3. It was suggested that an effort be made to remove Boston from Zone 3.
4. A review of Algermissen map and the assumptions on which it is based was suggested.
5. Critical evaluation of historic records (not to discount damage of 1755 Cape Ann earthquake).
6. Resolve the question of the epicentral intensity of the 1755 event - Linnehan of Weston Observatory has estimated an epicentral intensity of MMI 7. Richard Holt of Weston Geophysical has estimated an epicentral intensity of MMI 8. Felt intensity in Boston for the 1755 event have been estimated at MMI 5 to MMI 6. The Ossipee, N.H. earthquake of 1940 had an epicentral intensity of MMI 7. It was noted that New England earthquakes don't relate to surface faulting.
7. It was suggested that a map of bedrock accelerations throughout the state be developed.
8. Prof. Whitman presented a review of SDDA study of seismic design considerations for high rise buildings on firm ground in Boston. Loss estimates developed in the SDDA study suggest that so little damage (in dollars) is expected that any additional code requirements may not be justified economically.

   Prof. Whitman recommended the following goals for the committee:

   A. Recommend to the State Building Code Commission the appropriate UBC seismic zones for Massachusetts.
   B. Recommend appropriate modification of BOCA/UBC.
   C. Recommend corrections for the apparent ambiguities in the BOCA code.
   D. Write code provisions in such a way that they can be modified as information improves.

9. It was pointed out that precast buildings have demonstrated negligible seismic resistance in recent experience.
10. Information was requested concerning the effect of local soil conditions.
11. The question was raised as to why the State Building Code is based on BOCA rather than UBC.
12. Inquiry was made as to the basis for current code provision with respect to seismic effects. Prof. Whitman responded that the basic guideline for the California seismic provision was as follows.

    For the maximum likely earthquake: no damage.

    For the maximum probable earthquake: no structural damage.

    For the maximum possible earthquake: no building collapse.

13. It was decided that the full committee should continue to meet as a group until its goals and scope of work were better defined.

    Information requested by the first meeting:

    1. Applicable sections of the BOCA code and the Uniform Building Code.
    2. Material on the seismicity of Massachusetts (from Fr. Linehan).
    3. Information on the effects of local soil conditions (from Prof. Whitman).
    4. Summary of Seismic Design Decision Analysis research at M.I.T. (from Prof. Whitman).
    5. Insight into the legislative process relative to the proposed code (from the Code Commission and Code Commission Chairman, Charles Theodore).

<u>October 5, 1973</u>

Prof. Whitman distributed to the Committee:

1. Description of proposed SDDA interaction with local engineers on seismic design question. (From M.I.T. proposal to National Science Foundation (RANN))
2. Internal Study Report on Seismic Risk in Boston. (See Seismic Design Decision Analysis Internal Study Report, "Analysis of the Seismic Risk of Firm Ground for Sites in the Central Boston Metropolitan Area," by H. Merz and C.A. Cornell, January, 1972.)
3. Notes on the effects of local soil conditions.

<u>October 15, 1973</u>

Second meeting of the ASCE/BSCE Joint Committee on Seismic Design Criteria. Charles Theodore of the State Building Code Commission became a member of the Committee.

Information presented:

1. Dr. Zaldastani distributed the relevant sections of the BOCA Code 1970 and supplements for 1971 and 1972 (which do not include ductility requirements).
2. Mr. Kaye of USGS reported that Dr. Algermissen is preparing a new seismic risk map of New England, and distributed a map of Massachusetts showing deposits of fine grain soils.
3. Mr. Holt of Weston Geophysical distributed an epicenter map of instrumentally determined earthquakes in Massachusetts and a supplemental tabulation of earthquake data.

There was discussion on:

1. Base rock motions.
2. Effects of local soil conditions.
3. Return periods of earthquakes in Massachusetts.
4. Ductility requirements and provisions for resistance to lateral forces.

Decisions:

1. There was consensus in the committee that it is inconsistent to design buildings in Massachusetts in accordance with requirements established for California. It was agreed that SEAOC Zone 3 seismic requirements are not reasonable for Boston. Because of the differences of seismilogy and geology between Massachusetts and California there is no rationale for using the same seismic design criteria. Reasons may be developed for not recommending change in the current seismic risk zone designations for Massachusetts. (Zone 2 BOCA, with no ductility requirements)
2. It was agreed that with low seismic risk, investment in seismic design could not be justified as insurance to avoid future damage. The only legitimate justification was seen to be the life safety consideration.

The Committee requested further information:

1. A presentation of the methodology of handling ductility requirements and their implications.
2. A comparison of practice in Boston and West Coast cities regarding design wind and snow loads.
3. A comparison of seismicity in Massachusetts with sites in Zones 2 and 3 elsewhere in the country.

Further questions arose on the relationship of lateral force requirements and ductility requirements. Does an increase in the lateral force requirement imply an increase in ductility?

October 29, 1973

"Seismic Design Decision Analysis" paper presented to the American Society of Civil Engineers Annual and National Environment Meeting in New York City. (See Whitman, R.V., J.M. Biggs, J.E. Brennan III, C.A. Cornell, R.L. de Neufville, and E.H. Vanmarcke, "Seismic Design Decision Analysis" Journal of the Structural Division, American Society of Civil Engineers, May 1975.)

November 1, 1973

Third meeting of ASCE/BSCE Joint Committee on Seismic Design Criteria.

Reports:

Mr. Theodore explained that the new code was to be ready by July 1, 1974 and promulgated on July 1, 1975. At that time BOCA 1970 was the minimum seismic requirement for new construction in the state. However, for practical purposes these requirements were not enforced outside of Boston. The Commission did not consider this situation adequate. It did not believe it appropriate to adopt the same provisions as California. The Commission would probably adopt what the Committee recommended, but it did not wish to adopt requirements that would seriously escalate the cost of construction in the Commonwealth.

Prof. Holley suggested that the Commission might benefit from a statement by the Committee on how to deal with the lower level of seismic risk evident in Massachusetts. Professor Holley speaks for a selective applicationtion of seismic provision which would identify "fragile" building types and preclude such constructions under certain combinations of the following variables:

a. geographical location within Massachusetts, to the extent that this is related to the magnitude of the earthquake event;

b. soil conditions at the site;

c. function of the building (hospital, for example);

d. density of building occupancy, i.e. number of occupants at risk.

This approach focuses on avoiding hazard to life rather than risk of damage to buildings. (See "A Possible Approach to Building Code Provisions for Seismic Loading in Massachusetts" by M.J. Holley, Jrl, November 1, 1973)

On the basis fo material provided by Mr. Souza, it was concluded that Boston structures were not over-designed for wind and snow load when compared to California; however, higher calculated wind loads would provide Boston buildings with some reserve strength compared with California buildings designed for the same seismic zone criteria.

Fr. Linehan indicated that the Zone 3 designation for Boston on the USGS map was based on very tenuous data and currently is undergoing revision.

Prof. Cornell distributed data on frequency of seismic activity in the United States and Canada. The frequency of seismic events is about 20 times greater in California and Nevada than in New England, but the rates of attenuation of energy in the East are lower than in the West.

Discussion:

Discussion in this meeting recognized the need for an independent approach to the problems presented in areas of relatively lower seismicity. There is a need for an approach which can balance costs and risks more effectively. A strategy must be developed which will reduce risk at minimum cost. Emphasis may be placed on improvement of construction details, limiting of parapets, improvement of ductility rather than simply increasing lateral force requirements.

Actions:

A steering committee was formed of Whitman, Holley, Simpson and Zaldastani to divide the Committee into two groups:

A. Soils/Seismology

B. Structures.

Each subgroup was instructed to consider:

1. Determination of what circumstances of geographic location, subsurface conditions, type or detail of construction or occupancy warrant special

attention to seismic resistance in the new State Building Code.

2. Formulation of provisions to insure necessary minimum seismic resistance in such instances.

November 27, 1973

"Summary of SEAOC Proposed Code to Include Site Soils Effects in the Lateral Force Equation for Seismic Design," from the Geotechnical Subcommittee of SEAOC was distributed to the Soils/Seismicity subcommittee. This is a recommendation for adding "s" a soil factor and modifying "c" the coefficient for base shear in the lateral force equation for seismic design.

Also a "Draft Recommendation on Soil-Structure Interaction Effects" from SEAOC to UBC was distributed. This relates the natural period of the structure to the period of the soil and takes account of possible greater intensities on poor soil.

December 28, 1973

Questions to be considered by Sub-sub-Committee A1 (Seismology) of Task Group A (Soils/Seismology):

1. It is reasonable to assume that a repetition of the 1755 earthquake might cause ground accelerations as high as 0.1 g in the vicinity of Boston.
2. Sketch contours (in MMI and peak acceleration) of the intensity on firm ground of an event with 100 year return period.
3. Sketch contours (in MMI and peak acceleration) of intensity on firm ground of an event of 10,000 year return period.

It is at this point that discussion began to focus on the use of a design earthquake with a peak acceleration of about 0.1 g. The development of this concensus facilitated continued subcommittee deliberations.

January 2, 1974

Meeting of Sub-sub-Committee A2 (Soils)

Goals for the Sub-sub-Committee A2 (Soils) of Task Group A (Soils/Seismology):

1. To identify soil conditions that might increase seismic (dynamic) forces on structures by 100% or more, as compared to forces in the same buildings on firm ground.
2. To suggest code provisions to identify such conditions.
3. To identify areas where ground might fail (spread or settle) during an earthquake ground motion with a peak acceleration of 0.1 g.

Agreement was reached on priority of the following:

1. A definition of firm ground.
2. Identification of areas where the ground might fail by spreading or settling during an earthquake.

It was concluded that futher input was required from the Sub-sub-Committee on seismology on the <u>design earthquake</u> for firm ground

Dr. Castro submitted further consideration of the question of "resonance" conditions of building and ground. Evaluation of possible resonance would require specific analysis in each case, i.e. determination of moduli for all soil strata and a dynamic analysis for the building. Dir. Castro concluded that the Massachusetts code should not require such analysis, though there may be special consideration for buildings on "non-firm" ground and for critical buildings higher than 10 stories.

Consideration was also given to the basic approach to code formulation. The Sub-sub-Committee discussed towhat extent the Massachusetts Seismic Provisions should be a rewrite or extension of BOCA. It was recommended to adopt guidelines for cases in which the "escape clause" (719.1 Exemptions) would <u>not</u> apply. These guidelines may include such conditions as unusual structure, occupancy, soil, or seismicity.

<u>January 3, 1974</u>

Letter to Members of Joint Committee on Seismic Design Criteria. Continued work was carried out in two Task Groups.

Task Group A (Soils/Seismology)

The Assigned Task of Group A is to agree on maximum acceleration criteria for regions throughout Massachusetts. It will also flag and recommend criteria for any special circumstances where the nature of the soil conditions and/or location is such as to require special consideration.

Task Group B (Structures)

Task Group B initially must decide on the minimum design criteria to be imposed on structures whose occupancy, type, or location is such as to require attention to their seismic resistance (including that of secondary elements and their attachments). It is suggested to assume as a starting point that the 1755 earthquake caused a maximum ground acceleration of 0.1 g in the Boston area (corresponding to ground motions about one-third those measured in the 1940 El Centro earthquake).

<u>January 18, 1974</u>

Meeting of Sub-sub-Committee A2 (Soils) of the Task Group A (Soils/Seismology).

Discussion:

There was extensive discussion of the influence of local soil conditions on structures, including consideration of soil amplification, differential settlement, spreading and liquefaction of foundation soils. It was then speculated that most shallow deposits of soft soil will probably increase peak ground surface accelerations by a factor of two relative to firm ground. (This opinion has since changed.) Consideration should be given to foundation details to minimize lateral spreading and differential settlements.

Prof. Whitman discussed the effect that local soil conditions can have on the coefficient "c" in the UBC formula for computing seismic lateral force. Prof. Whitman recommended review of the SEAOC paper, "Effects of Local Soil Conditions upon Earthquake Ground Motions," and the consideration of a soils factor for the Massachusetts Code.

Information distributed:

The results of M.I.T. (SDDA) site amplification studies were distributed to the Committee members.

January 23, 1974

Meeting of Task Group B (Structures)

Dr. Simpson proposed the preliminary adoption of a maximum ground acceleration of 0.1 g for the Boston Area, allowing for subsequent adjustment by Task Group A. Dr. Zaldastani recommended that attention be given to the form that results of Committee deliberations might take. The results had to be in the form of usable code provisions and had to be available by April for final revision and submission in July.

The point was made that heavy reliance would have to be made on the following available code resources:

1. Structural Engineers Association of California, SEAOC - provides the most advanced source in this country but applies to Zone 3 only.
2. Uniform Building Code, UBC - relies on SEAOC and provides provisions for all zones.
3. Building Officials Conference of America, BOCA - relies on UBC, but follows with a significant time lag.
4. American Concrete Institute, ACI - limited scope, concrete, Zone 3.
5. Portland Cement Association, PCI - is limited to design criteria for concrete construction.

Dr. Zaldastani suggested using SEAOC as a pattern and he suggested the preparation of a master chart comparing the basic requirements in the available codes.

Decisions:

As the Massachusetts State Code would be based on BOCA, it was decided to study what changes would be needed in BOCA to incorporate the provisions which UBC makes for Zone 1.

February 7, 1974

Meeting of Task Group b (Structures)

Reports:

Mr. Theodore presented the format of the new State Code. It is to be based on BOCA format with ACl 318-71 referenced.

Professor Whitman reported on a recent meeting of the Applied Technology Council Seismic Design Review Group. The Applied Technology Council, Seismic Design Review Group has agreed that code concern should be minimum of life loss.

SEAOC recommendations, dated December 10, 1973, for changes in UBC were distributed.

Decisions:

The comparison of model codes was continued and decisions were made on what to retain and what to change.

The task group decided to prepare recommendations for Zone 1 and implement other zoning recommendations as indicated by Task Group A.

It was mentioned that wind loads may govern in Zone 1, except in the case of certain interior walls.

Dr. Zaldastani will prepare a composite of applicable BOCA sections and references for Committee review.

The following sub-committees of Task Group B were formed:

1. Ductility requirements for concrete and steel;
2. Loading;
3. Special provisions;
4. Precast connections.

It was not clear at this point which Task Group would evaluate intensity vs. design criteria.

February 13, 1974

The variables to be considered by the Committee include:

1. Probability of earthquake occurrence
2. Frequency content, intensity, and duration of shaking of the input motions at rock.
3. Fundamental period of the overlying soils and amplification effects
4. Fundamental period of the structure.

Decisions:

The SEAOC recommendation for changes in sections 2314(c) and (d) of the UBC were studied and discussed by the Committee. It was voted that no specific soil factor is included in the proposed changes to account for soil amplification effects.

It was proposed that dynamic analyses be undertaken to investigate soil amplification effects for various deposits of soft soil and for various earthquake inputs. These analyses would be conducted to ascertain whether or not it is feasible to construct an envelope curve of soil amplification versus site period for the range of soft soil profiles believed to exist in Massachusetts. If successful, the results of such analysis would be used to assist in formulating code provisions for soil amplification effects. (These studies were carried out by the Seismic Design Decision Analysis Project at M.I.T. See M.I.T., SDDA Internal Study Report 15, "Soil Amplification Studies for Typical Soil Profiles in the Boston Area, J.N. Protonarios, Sept. 1972.)

Further research on Boston Selectman's records and Ipswich town records of 1755 and 1756 would be undertaken to determine whether more detailed information is available on the 1755 earthquake.

Code criteria for soil amplification and site-building resonance will also be developed.

<u>February 27, 1974</u>

Draft of 719.0 Earthquake Regualtions was distributed by Dr. Zaldastani. Based largely on UBC Zone 1.

<u>March 1, 1974</u>

Status of Task Group A (Seismology/Soils) thinking as of March 1, 1974, by Prof. R.V. Whitman.

Seismology: A set of maps is being prepared giving zones of "expected" intensities for 50 year, 100 year, and 1000 year return periods and the maximum credible event (AEC safe shutdown). Until such maps are produced three zones may be considered corresponding to the following modified Mercalli intensities:

	Zone A	Zone B	Zone C
100 year quake	V	VI	VII
1000 year quake	VI	VII	VIII
max. credible	VII	VIII	IX

Where Zone C is appropriate for Cape Ann, Zone B for the rest of eastern Massachusetts, and Zone A for the rest of the state.

Soils: A definition of firm ground has been written which suffices to permit correction of risk maps to firm soil. Soil effects on base shear factors have been discussed. Most likely, some increase (50 - 100%) in the base shear coefficient for soft soils will be recommended, at least where tall buildings are founded over deep soft soils. Consideration has also been given to identifying combinations of intensity and soil conditions which might lead to liquefaction or slope failure.

Seismic Design Decision Analysis:

The M.I.T. staff of the SDDA project planned to assemble information concerning the implications of adopting various design requirements. This would take the form:

Design Level	Zone A	Zone B	Zone C
No requirements			
1/4 SEAOC			
1/2 SEAOC	(Expected Annual Life Loss)		
SEAOC			

March 5, 1974

Letter form Prof. Whitman to Dr. Simpson.

Professor Whitman commented on the February 27 draft of earthquake regulations. He made specific suggestions regarding the calculation of "C" ($C = 0.075/\sqrt{T}$)and "T". He recommended the inclusion of examples of structural systems for various K values. He supported the encouragement of dynamic analysis. He suggested restoration of the special provisions of Zone 2.

March 6, 1974

SDDA project begins preparation of seismic risk contour maps based on assumptions of rectangular source zones, upper bound events, and rates of attenuation.

March 7, 1974

Background for Soil Factors prepared by Prof. Whitman.

(This draft was an outline of approaches and ideas rather than a suggestion of code wording)

Soil factor $\underline{s}$ should be a multiplication factor in the equation for base shear.

- $$s_0 = \begin{cases} s_0 \text{ for } & T \leq T_0 \\ s_0(z - T/T_0) + (T/T_0 - 1) & \text{for } T_0 \leq T \leq zT_0 \\ 1 \text{ for } & 2T_0 \leq T \end{cases}$$

- T_0= H/150 where H is the depth in feet to firm ground below footing, pile, caisson caps, or mat.

- s_0= 1 for firm ground
  = 1.5 for soft ground.

Special provisions for soft soil (Massachusetts assumed to be one zone):

- All footings, pile and caisson caps must be tied together as in UBC
- Ductile construction must be used
- Special site investigation to assure adequacy against liquefaction or slope instability.

March 7, 1974

"Background for Code Soil Factors" by Prof. Whitman

Review of damage vs. soil vs. types of construction.

If a building is to resist earthquakes on soft ground,

1. it should be of ductile construction
2. the foundation should be tied together
3. the site must be safe from liquefaction or stability failure.

Review of soil factors vs. depth of soil and period of building.

A stiff structure on deep soft soil may be less likely to yield, but if it does yield, the inelastic distortion will be increased.

Choice of specific soil factors.

For Massachusetts, it seems reasonable to think of just two categories of ground, soft and firm. There may be ambiguity in the case of cohesionless soils, but the simplificateion seems worth the ambiguity. The value of s_0=1.5 for poor soil is rather arbitrary, though it has been used in California, and appears right. The rule T_0=H/150 is a reasonable fit to theoretical calculations for Boston. The question of resonance in deep firm soils is still debated.

A table was presented relating building period to soil conditions. The table indicated effects for a stiff building ($T \approx 0.3$ sec.) and a flexible building ($T \approx 1.5$ sec.) for shallow soft soil (50 ft.) and deep soft soil (200 ft.).

Also a table was presented relating spectral acceleration - g to building period - seconds

Results of SDDA Risk Analysis:

	MMI			
Return period, yrs.	Cape Ann	Boston	West	North-Mid State
100	6.0	5.0	5.0	4.9
1000	7.5	6.7	6.5	6.0
Max	8.7	8.7	8.3	7.5

There continued to be discussion on whether these numbers should apply to soft ground or firm soil. These values were assumed to hold for soft soil.

March 15, 1974

Memorandum to Task Group B on proposed code provisions in relation to the level of risk from Prof. Holley.

Thus far Task Group B has devoted its efforts to a review of existing (BOCA) code provisions, with the thought that earthquake provisions in a Massachusetts Code might be similar in form. It was recognized that, reflecting recommendations to be received from Task Group A, we might wish to modify the specific values of numerical coefficients to reflect the Massachusetts earthquake risk and, possibly, to account for such factors as variability of risk across the Commonwealth, local soil conditions, density of occupancy, and importance of the building function. To date, however, we have not addressed these latter questions, but have concentrated on familiarizing ourselves with the existing (BOCA) provisions, and how they might be re-stated for greater clarity. This has been a necessary, and valuable, exercise. However, we now must consider:

a. whether the BOCA approach is suitable for the Commonwealth of Massachusetts;

b. if suitable, what modifications are appropriate? If not suitable, what alternative?

Damage vs. Collapse

The BOCA provisions were evolved in the context of the West Coast seismic threat; i.e., frequent small to moderate earthquake events and infrequent large events. Thus limiting damage from small events was at least as important an objective as avoiding collapse during a large event. In Massachusetts the risk (of small as well as large earthquake events) is at least an order of magnitude smaller than on the West Coast. In view of this difference, it can be argued that in Massachusetts, increased building construction costs reflecting code provisions directed toward reduction of earthquake damage costs are not economically justified; and earthquake provisions in a Massachusetts building code should be focused upon the earthquake threat to human life, that is, upon the hazard of building collapse.

The Threat to Human Life in Relation to Evolving Technology

It should be kept in mind that we are in a period of active growth in our knowledge of the earthquake hazard, and in the technology for confronting this hazard. Within the next few years there may well be substantial changes (e.g. wider use of computer-implemented dynamic analysis, improved earthquake forecasting, improved understanding of structural behavior, more rational approaches to societal investments toward mitigation of earthquake hazards), and these advances may well lead to substantial changes in building codes. If a severe earthquake occurs in Massachusetts during this same period (the next 5 to 10 years) the hazard to human life in consequence of building collapses will reflect far more those buildings which already exist than new buildings constructed during the same period. From this it can be argued that earthquake code provisions introduced in Massachusetts at this time should be recognized as interim provisions only, and should be focused upon buildings functionally vital to post earthquake recovery rather than upon the total stock of new buildings.

A Possible Approach for the Massachusetts Code

In what follows it is assumed that Task Group A will confirm that Massachusetts is an area of relatively low earthquake risk. It is further assumed that Task Group A will identify a few local zones of higher risk than the rest of the state, with som of the Group believing that the risk is essentially constant across the entire state. Based on these assumptions and all the foregoing discussion, the following approach is offered for the Committee's consideration:

° Identification of a subset of buildings for special attention in the code. This subset would include those buildings whose function is vital to post earthquake recovery, wherever they are located in Massachusetts. It may be desirable to include certain buildings not essential to recovery, but for which the loss of life resultant from collapse would have a particularly devastating psychological impact. In this category the only obvious candidates are the public schools.

° For all buildings throughout the state not falling within the identified subset either

 i. do not require any provisions for seismic design

 or ii. require only the minimum (Zone 1) provisions of the present code.

 However, present provisions should be supplemented (even in the initial version of the Massachusetts Code) with provisions which

 i. address the problem of construction on poor soil

 ii. address the problem of inherently non-ductile elements and details, either by excluding their use on the vital building subset or by requiring their design for factored forces.

Non-ductile Elements

If building collapse is not to occur, there must be no failure of elements which are essential to overall system stability. Inelastic excursions of individual elements are admissable, and even advantageous, provided that system deflections do not become so large as to threaten overall stability. There are, however, certain elements which are either inherently non-ductile or not reliably predictable in their degree of ductility. Failure of such an element may couse collapse of a system which otherwise would have, in combination, sufficient strength and ductility to resist a severe earthquake. Such elements should be either excluded, or re-detailed to achieve the necessary ductility, or overdesigned to exclude demands on their limited ductility. An example of required re-detailing for increased ductility is the set of provisions for ductile R/C construction. These provisions reflect tests which provided the necessary bases for the recommended details. One can cite other examples for which the necessary underlying tests have not yet been performed. It may be necessary in such cases either to exclude the element or to require substantial overdesign. Either of these approaches is likely to stimulate the necessary test programs.

It is not realistic to expect that the first version of the Massachusetts Code can address all structural elements and details which are, of may be, hazardous in their limited ductility. In future versions of the Code there will be an opportunity to address additional elements and, on the basis of new evidence, it may indeed be desirable to relax earlier provisions.

March 19, 1974

Preliminary Draft of Proposed Earthquake Regulations

From Dr. Zaldastani to be reviewed by members of Task Group B.

March 27, 1974

Preliminary recommendations of sub-sub-committee A2 (Soils)

1. Definitions of firm ground, soft ground
2. Instability of soils
3. Soil factor for calculation of base shear to modify the UBC formula to $V = ZKC\underline{S}W$
4. Interconnections of foundations
5. Interpretation of seismicity studies recognizes that intensity curves of the sub-sub-committee A1 (Seismology) are based on unaltered data for natural soft soils, plotted intensities will be one level lower for firm ground.

March 27 1974

Massachusetts Seismic Risk Maps presented to the Committee by Prof. Cornell.

The maps indicate where the maximum epicentral location is more likely to occur and what expected frequency of various levels of intensity at any site in the state.

The first step in the risk analysis is to identify source areas. Four such areas have been used:

A. Across Connecticut, Rhode Island and southeastern Massachusetts, with maximum epicentral intensity 8.3.

B. Along western border of New England, also with epicentral intensity 8.3.

C. A source with the maximum epicentral intensity 8.7, but having three possible trend lines E-1, C-2, C-3. All versions include the Cape Ann area. C-1 and C-2 are considered more likely than C-3.

D. All other areas with a maximum epicentral intensity 6.3.

Within each source an earthquake is equally likely to occur at any point. Then recurrence rates for each source, and a law giving attenuation of intensity with distance, are determined from the historical record. All of this information is then combined together in a computer program that determines the rate at which various intensity levels are experienced at different points in the state. Risks are calculated for sites on a 25 mi^2 grid for intensities 4.0, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5 and 9.0.

The material was presented in the form of graphs and contour maps. The curves presented annual risk of equalling or exceeding intensity vs. intensity for four sites in the state: (See Fig. 1, Page A-23)

1. Cape Ann
2. Boston
3. South midstate
4. Far west state.

The contour maps presented isoseismals for MMI 5,6,7,8 for return periods of 100, 1000, 10000 and 1,000,000 years. (See Fig. 2, Page A-24)

March 27, 1974

Evaluation of reduction of seismic risk with reduction in projected life for buildings in Boston.

Current thinking of the ASCE/BSCE Seismic Committee was that Massachusetts should be Zone 1.5, the expected peak ground acceleration should be 0.12 g. In addition for buildings on soft soil a soil amplification factor (1 to 1.5) should be used. It was computed by Rene Luft that decreasing the design life of a structure from permanent (100 years) to 25 years corresponds to a reduction of 0.8 on the MMI scale and reducing the life to 10 years corresponds to a reduction of 1.5.

Building Life (Years)	Equal probability of maximum intensity experienced (MMI)
100	7.5
25	6.7
10	6.1

March 27, 1974

Summary of earthquake risk in Boston by Prof. Whitman

The maximum Richter magnitude for New England has been estimated to be a low 6 (6.2). Any such earthquake is expected to have a focal depth of about 20 to 25 km, and should not cause surface faulting. These estimates are based upon the maximum size of New England earthquakes during the past 400 years. According to the Gutenberg-Richter standard correlations, an earthquake of magnitude 6.2 would correspond to an epicentral modified Mercalli intensity VIII and to peak ground accelerations of 0.15 g to 0.20 g. However, the intensity of shaking in the epicentral region is influenced greatly by the actual depth of focus and the presence of soft soils. It seems reasonable to conclude that the earthquake of magnitude 6.2 would cause an epicentral intensity between VIII and IX if soft soils are present.

Table 1

Earthquakes with Magnitude About 6

Earthquake	Magnitude	shallow focus or surface faulting*	soft soils*	epicentral intensity
Long Beach, California (1933)	6.3	X	X	IX
Helena, Montana (1935)	6.0	?	X	VII-VIII
Ossipee, N.H. (1940)	5.8	-	-	VII
San Francisco, California (1956)	5.3	X	-	VII
Skopje, Yugoslavia (1963)	6.0	X	X	IX
Parkfield, California	5.5	X	-	VII
Southern Illinois (1968)	5.5	-	-	VII
Banya Luka, Yugoslavia (1969)	6.3	-	X	VII-IX
San Fernando, California (1971)	6.6	X	-	IX-X
Managua, Nicaragua (1972)	6.3	X	-	X

* causes more severe damage

Intensity-Risk Study (Summary of the material presented by Prof. Cornell to the Committee on March 27, 1974)

The study indicates where the maximum epicentral location is more likely to occur and what expected frequency of various levels of intensity at any site in the state.

The intensities in Table 2 should be interpreted as the intensity on poor soil. Intensities on firm ground would be less, although it is difficult to say how much less. A reduction of one unit is good general guidance although this reduction possibly should be greater for the maximum intensity and less for the smaller intensities associated with 100 and 1000 year return periods.

Table 2

RESULTS OF RISK ANALYSIS

Return period years	Modified Mercalli Intensities Cape Ann	Boston	West and South	North, mid-state
100	6.0	5.0	5.0 (5.4)	4.9
1000	7.5	6.7	6.5 (7.0)	6.0 (6.3)
Maximum	8.7	8.7	8.3 (8.7)	7.5 (8.0)

Intensity vs. Damage

Table 2 summarizes the relationship between intensity and damage, based upon the definition of the modified Mercalli scale.

Table 3

DAMAGE VS. INTENSITY FROM MODIFIED MERCALLI SCALE

(masonry and frame construction)

(for firm soil, add one intensity level for soft soil)

Type of Building	MMI VI	VII	VIII	IX
Poorly built or badly designed	Damage	Considerable	many collapse	most collapse
Well-built ordinary buildings	Plaster cracks	slight to moderate	considerable; some collapse	great; some collapse
Good design and construction (designed to resist earthquakes)	-	negligible	slight	considerable

The SDDA study has proposed damage matrices for reinforced concrete frame and shear wall buildings, based on a combination of experience during actual earthquakes and theoretical analyses. The following worksheet summarizes the findings. (This worksheet applies to firm soil; add one intensity unit for soft soil. Thus the column head VIII applies for the maximum intensity that might occur (with a low probability in eastern Massachusetts)). Both experience and theoretical analysis indicate that design for "UBC Zone 1" has little beneficial effect over no specific earthquake design (from experience in Caracas, 30-40 miles from epicenter of magnitude 6.3 earthquake).

Table 4

For reinforced concrete frames and shear walls; little or no attention to seismic design of non-structural elements; Boston level of wind design used.

Design Strategy	MMI Ground Shaking that Occurs on Firm Ground			
	VI	VII	VIII	IX
No Seismic Requirement	3/4 have light damage	most have damage 1/3 moderate 5% heavy	all damaged; 1/3 total loss; 5% collapse	1/4 collapse; all total loss
"Zone 1"	SAME AS WITH ZONE 0 DESIGN			
"Zone 2"	1/2 have light damage	3/4 damaged 1/3 moderate 1% heavy	all damaged 1/3 heavy 4% total loss 1% collapse	1/5 collapse; all total loss

Some data are also available for steel buildings, indicating somewhat more damage at low intensitites and less likelihood of collapse at high intensities. The information for various types of construction are summarized on the following page in Table 5, where the intensities correspond to those in Table 2 and on the risk maps.

Table 5 caused difficulty for many committee members but once the relationship of resulting damage for similar intensities on firm and soft ground was understood, it gained acceptance. This table played a major role in helping the committee decide that Zone 1 design was inadequate.

Type of Construction	BUILDINGS ON FIRM SOIL				BUILDINGS ON SOFT SOIL			
	MODIFIED MERCALLI INTENSITY (WHERE BOTH FIRM & SOFT SOIL EXIST)							
	6	7	8	9	6	7	8	9
Ordinary Masonry Buildings	No Damage	Plaster Cracked	Slight to Moderate Damage	Much Damage; Some Collapse	Plaster Cracked	Slight to Moderate Damage	Much Damage; Some Collapse	Great Damage; Some Collapse
Masonry Buildings with Zone 2 Design	No Damage	No Damage	Light Damage	Slight Damage	No Damage	Light Damage	Slight Damage	Considerable
Framed Buildings with Zone 0 or Zone 1 Design	No Damage	75% Light Damage	85% Damaged 5% Heavy Damage	35% Total Loss; 5% Collapse	75% Light Damage	85% Damaged 5% Heavy Damage	35% Total Loss; 5% Collapse	25% Collapse; All Total Loss
Framed Buildings with Zone 2 Design	No Damage	50% Light Damage	75% Damaged 1% Heavy Damage	35% Heavy Damage; Perhaps 1% Collapse	50% Light Damage	75% Damaged 1% Heavy Damage	35% Heavy Damage; Perhaps 1% Collapse	10% Collapse; Most Total Loss

Table 5

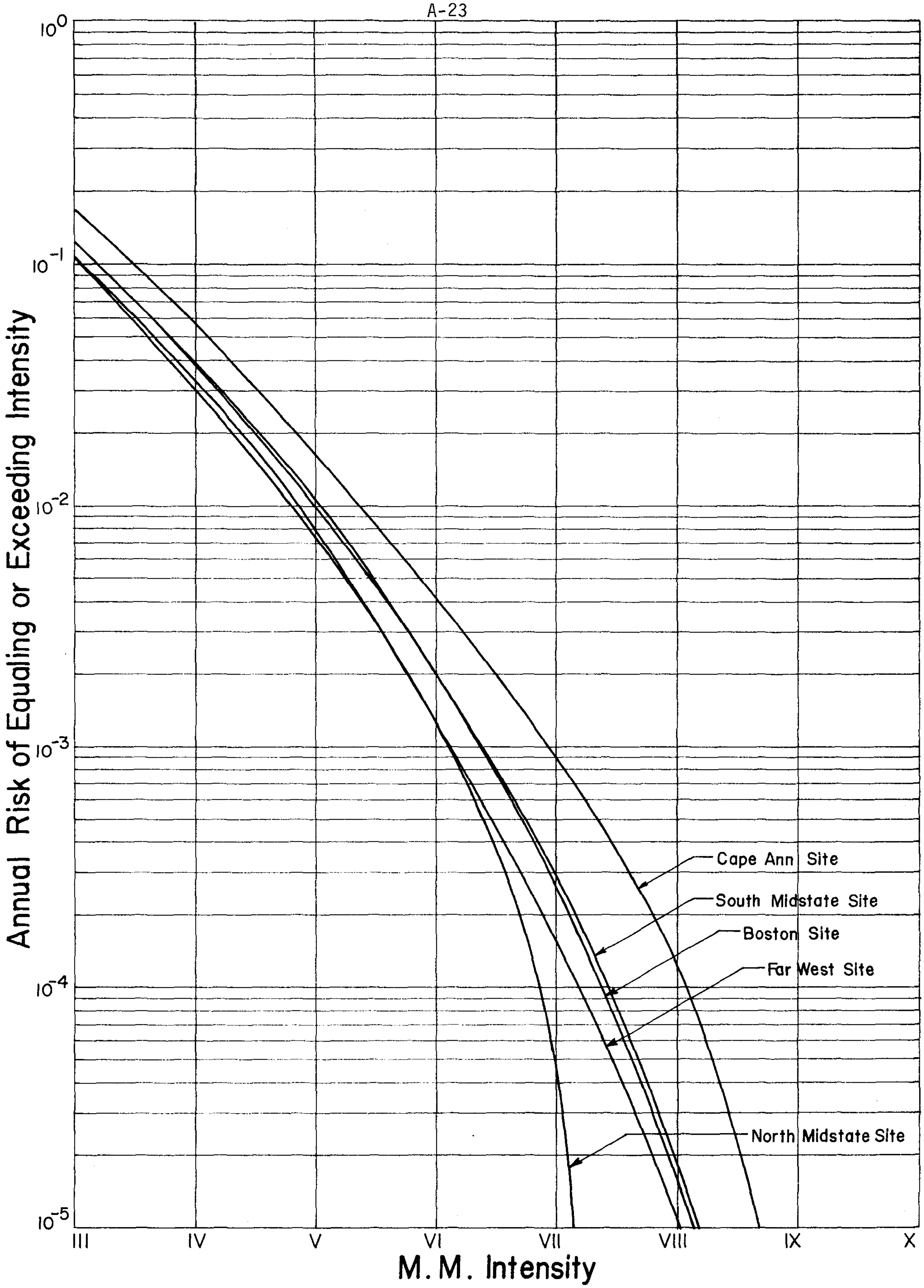
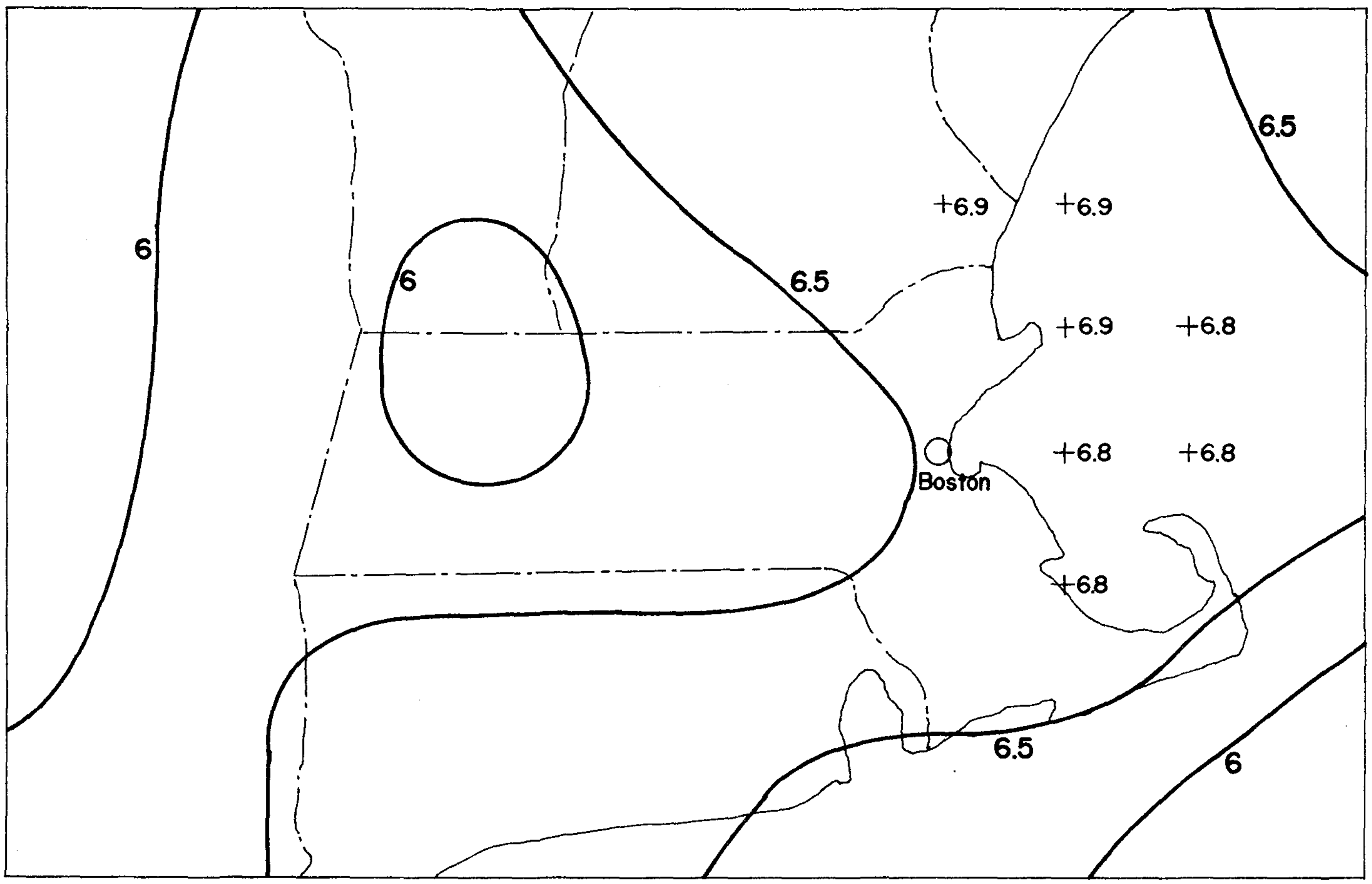

10^0
10^{-1}
10^{-2}
10^{-3}
10^{-4}
10^{-5}
Annual Risk of Equaling or Exceeding Intensity
III
IV
V
VI
VII
VIII
IX
X
M. M. Intensity
Cape Ann Site
South Midstate Site
Boston Site
Far West Site
North Midstate Site

Figure 1



M.M. Intensity Contours: 1000 Year Return Period

Figure 2

March 1974

Choice of Design Criteria based on small likelihood of collapse during largest earthquake.

Firm Ground

Sub-committee A's recommendations all add up to the maximum earthquake having intensity VII to VIII on firm ground (might have peak acceleration of 0.15 g). Such shaking as at the threshold point for damaging engineered buildings:

° Modern buildings in Los Angeles had beginnings of structural damage at this intensity

° There was partial collapse of a few old steel buildings in Los Angeles at this intensity

° Modern buildings in Caracas (Zone 1 1/2 design) were structurally damaged at this intensity, and there were some collapses with intensity of about VIII.

Conclusions

The maximum expected intensity falls in a critical range where damage and danger of collapse begins, and definite statements about possibility of collapse are just not possible.

1. If one wishes to be conservative, the ductility and lateral force requirements for Zone 2 should be enforced.*
2. If one wishes to be less conservative, no seismic provisions could be justified.
3. Zone 1 provisions re-cladding and masonry walls will help but lateral force requirements will have only marginal benefit.

Soft Ground

The effects of soft ground are

1. Foundation failures (commence being a problem when firm ground MMI > VII)
2. Differential foundation movements (also commence being a problem when firm ground MMI > VII)
3. Increase longer period components of shaking, thus increasing:
   a. Likelihood that more flexible buildings will yield
   b. Necessity of ductility in buildings that have yielded

Conclusions

If considering the maximum firm ground intensity of VII 1/2, should:

1. have provisions requiring analysis of really marginal land.

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* Alternative might be to require larger lateral forces and omit ductility requirement.

2. require tieing together of footings or pile caps resting on soft ground.
3. require ductility in buildings founded over soft ground. (Alternative might be to require larger lateral forces and omit ductility requirement.)
4. if lateral force analysis is required, introduce soil factor.

<u>March, 1974</u>

Base shear and ductility requirements in SEAOC by Dr. Rene Luft.

See Seismic Design Decision Analysis, Reoprt No. 4 "Seismic Response of Buildings Designed by Code for Different Earthquake Intensities," by Prof. J.M. Biggs and P.H. Grace.

Dr. Luft noted that for many Boston buildings, in comparing lateral force requirements for wind load and seismic lateral force requirements for Zone 1 UBC, that wind load would govern. There is an indication that buildings designed for Zones 2 and 3 lateral forces have a lower elastic reserve than Zone 1 and therefore require stricter ductility requirements.

<u>March 28, 1974</u>

Memorandum from Prof. Holley to Dr. Simpson with philosophical questions for the Committee.

<u>Question 1</u> (regarding intensity of assumed earthquake event). What is the maximum earthquake intensity to be reflected by code provisions for the Commonwealth?

<u>Question 2</u> (regarding factors which increase the hazard for a particular building).

a. Shall the stringency of code provisions reflect the soil conditions at the site?
b. Shall the stringency of code provisions reflect the type of construction (inherently ductile vs. inherently brittle; regular vs. irregular building configuration, etc.)?

<u>Question 3</u> (regarding factors which relate to the consequences of adverse response of a building to an earthquake event).

a. Shall the stringency of the code provisions reflect population density of the area in which the site is located (city vs. suburbs vs. rural)?
b. Shall the stringency of the code provisions reflect the density of occupancy of the building in its normal function?
c. Shall the stringency of the code provisions reflect the importance of the building function to the immediate post-earthquake assistance to those who are injured?
d. Shall the stringency of the code provisions reflect the importance of the building function to the post-earthquake general recovery of the locale damaged by the event?

April 9. 1974

Danger of collapse vs. Intensity, and Performance of buildings not specifically designed for lateral forces, by Prof. Whitman.

Ratings of the strength of historical earthquakes is based upon the response of affected buildings. Such response is greatly affected by the quality of materials and workmanship. The MKS intensity scale references three types of essentially non-engineered construction:

a. Buildings in field stone, rural structures, adobe houses, clay houses.

b. Ordinary brick buildings, buildings of large block and pre-fabricated type, half timbered structures, building in natural brownstone.

c. Modern reinforced structures.

Tables were presented showing damage to non-engineered buildings by percentage of collapse vs. intensity (MSK).

Performance of engineered structures

Tables were presented from SDDA analysis of Anchorage and San Fernando earthquakes on damage to engineered structures by percentage of collapse vs. intensity. The data indicate that considerable damage can occur to modern engineered structues, supposedly having considerable seismic resistance, at higher intensities.

Comparison of the tables indicates that engineered structures do not necessarily perform better than non-engineered structures, presumably because they are typically larger and more complicated. Code requirements on lateral forces may be of relatively little help for such buildings; provisions aimed at ensuring ductility and dynamic analyses that examine the force paths through the building should be of more help.

April, 1974

Seminar on Risk Analysis for Boston Society of Civil Engineers by Professor Cornell.

April 29, 1974

Letter to Dr. Simpson from Mr. Kaye of U.S. Geological Survey.

The basic point to be determined by the Committee before any deliberations could be undertaken on code provisions is a "design earthquake," a probable earthquake. This is the critical decision. To make this decision, one has to weigh risk - an exercise that enters into all of life's decisions - and it is here that experience and wisdom, such as that the Committee possesses, counts. This is the exercise that distinguishes engineering design from non-engineering design. To insure safety against eternity is a pursuit better left to theologians.

For the state to insist on design for an earthquake with a 100,000 year return period is an act of social irresponsibility. (This is based on the hypothesis that an earthquake causing MMI VII 1/2 on firm ground in Massachusetts is an event with a 100,000 year return period.)

May 13, 1974

Preliminary Recommendations of Sub-sub-Committee A2 (Soils)

Definitions of Firm Ground and Soft Ground, Instability of Soils, Calculation of Base Shear (S factor) (assumes 0.15 g design acceleration on firm ground which corresponds to MMI 8.5 on soft ground).

May 1974

Applied Technology Council Recommended National Seismic Design Criteria.

ATC-3 Definition of Scope proposal submitted to National Bureau of Standards.

July 5, 1974

Summary, Earthquake Design Strategy for Massachusetts, by Prof.R.V. Whitman.

1. Aim:

The aim is primarily to ensure that there is an acceptably low probability of building collapse as a result of an earthquake such as the "design earthquake" defined in item 2 below. Full compliance with the words and spirit of this code plus sound engineering design as presented should ensure safety during the "design earthquake." In the absence of strict review and inspection and quality control, some buildings not fully meeting the spirit of these provisions may collapse.

These provisions are not aimed at eliminating damage short of total or partial collapse that will endanger lives. Owners may wish to require additional resistance so as to minimize probability of economic loss.

2. "Design Earthquake":

The design earthquake is defined as a ground motion as observed on firm ground, with a peak acceleration of about 0.10 g, a peak velocity of about 5 in/sec and a peak displacement of about 3 inches. This motion will be amplified by soft ground. In the code, this earthquake is represented by a series of response spectra for firm ground and for various depths of soft ground.

To some, this earthquake is the largest that is likely to occur in New England. Others believe that larger earthquakes may occur, but that use of this ground motion represents a reasonable level of risk. Given the current lack of knowledge concerning the mechanics of earthquakes it is not possible to state categorically the risk that the design earthquake might be exceeded.

3. Use of "Design Earthquake":

Any building is considered satisfactory if a dynamic analysis is performed and if either:

a. the computed forces, when combined with appropriate dead and live load forces, are less than the ultimate load capacities of members, or

b. it can be demonstrated, on the basis of tests or experience, that the building can tolerate the computed deformations without danger of collapse.

In addition, it must be shown, using appropriate tests or experience, that cladding, windows, partitions, ceilings and other non-structural features, whose falling might endanger lives, will remain in place under the computed deformations and motions.

This provision provides a basis for apporach of structural systems that do not qualify for the simpler methods of analysis described in item 4 below.

4. Simple Analysis Procedures for Ductile Structures:

Structures fulfilling the ductility requirements set forth in item 5 below may be analyzed and designed using the pseudo-static force approach in the Uniform Building Code. The lateral forces may still be as large as for Zone 1 times, for soft soil, a soil factor whose value varies with the depth of soft soil and which has a maximum value of 1.5. (The Committee later raised the lateral force requirement to Zone 1 1/2.)

These lateral forces are relatively small and in many cases design for wind will govern. The force requirement here will require that cladding and other exterior attachments will be well secured and that at least some attention is given to the strength of all connections. To a large extent, the safety of buildings depends upon the ductility requirements set forth in item 5 rather than upon the lateral forces stipulated in this item. For structues with periods near 0.5 sec, these lateral forces really are not adequate. Forthcoming changes in the Uniform Building Code will rectify this situation, and it seems better to wait for these changes rather than trying to introduce such changes ahead of time. (This refers to the change from $C = \frac{0.05}{\sqrt[3]{T}}$ to $C = \frac{1}{15\sqrt{T}}$

5. Ductility Requirements for Structures Qualifying for Simple Analysis Procedures:

The requirements applicable to Zone 1 of the Uniform Building Code are mandatory for buildings on firm ground, while the requirements applicable in Zone 2 are mandatory for buildings founded over soft ground. (The Committee later decided to require Zone 2 for all conditions.)

The requirements for soft soil, as outlined here, are more severe than those now in effect since every structue must either meet ductility requirements or be designed using dynamic analysis.

July 15, 1974

Preliminary Recommendations of Sub-sub-Committee A2 (Soils).

Definitions of Subsurface Conditions for Purposes of Seismic Design.

Instability of Soils and Rock.

Table 1 - Penetration Resistance Requirements for Sand Deposits Subjected to Earthquakes for Safety against Cyclic Mobility.

Table 2 - Permissible Thickness and Depths of Soils that are Susceptible to Liquefaction.

Calculation of Base Shear Force - Tables for determination of soil factor s.

Interconnections of Foundations Slabs-on-grade.

July 1974

Recommendations of Sub-sub-Committee A2 (Soils) for Seismic Building Code Provisions (Written in code language and numbered according to section)

718.2 Definitions
- Class A soil
- Class B soil
- Class A soil site
- Class B soil site
- Foundation level
- Liquefaction

718.41 Total Lateral Force

718.46 Lateral Force on Foundations

718.67 Interconnections of Foundations

718.68 Retaining Walls

723.1 Satisfactory Foundation Materials

723.3 Liquefaction

723.4 Class A and Class B Soils

723.42 Liquefaction During Earthquake

746.1 Surrounding Materials

Also recommendations on Slope Failure not related to earthquake:

723 Stability of Slopes

A dynamic analysis section 718.7, submitted separately, also reflects the effect of soil

718.4 Minimum Earthquake Forces for Structures

718.7 Dynamic Analysis, which allows for alternative ways to meet the requirements of section 718.

a. a dynamic analysis, based upon generally acceptable procedures, together with evidence showing that the building or structure can safely withstand the computed displacements and distortions.

b. a comparison of the building or structure with buildings or structues that have safely withstood a similar actual earthquake, or

c. other accepted procedures.

Figure 7-C Code Design Response Spectrum for the Design Earthquake giving spectral acceleration vs. fundamental period of structure.

Also a note for inclusion in 718.53 through 718.57 on non-ductile construction:

Buildings or structures must be capable of safely withstanding distortions 8 times those computed using the lateral forces specified in section 718.4.

<u>August 6, 1974</u>

Draft Summary of Ductility Provisions of the Body of Building Code Requirement for Reinforced Concrete (ACI 318-71)

by George H. Brattin, Portland Cement Association.

The ACI Code is a ductile code. Ductility was one of the five prime performance criteria upon which the code was developed. The intent is that the criteria of the body of the code is sufficient for areas where there is probability of light or moderate earthquake damage (i.e., Zone I and II). Changes have been made in the body of the code to increase ductility and the resistance of concrete structures to earthquakes and other catastrophic loads. Some of the more pertinent criteria are:

1. limitations on the reinforcement ratio for flexural members,
2. anchorage of positive moment reinforcement to develop yield,
3. hoop reinforcement on certain beam-column connections,
4. minimum shear and torsion reinforcement,
5. improvement in splicing and anchorage details,
6. provision for shift in moment.

This material was provided to support an argument that the provisions of the main body of the American Concrete Institute Code were adequate for New England and that the provisions of the ACI Appendix A (which UBC requires for Zone 2) are not essential.

<u>August 14, 1974</u>

Letter to Mr. Brattin from Ashly Gibbons, Codes and Standards section, Portland Cement Association.

Review and commentary on Mr. Brattin's Summary of Ductility Provisions of ACI 318-71.

<u>September 1974</u>

Proposal for Precast Concrete, Earthquake Section, Massachusetts State Building Code, from George H. Brattin, Portland Cement Association covering: Structural Elements
Exterior Elements

<u>November 14, 1974</u>

Applied Technology Council

ATC-3 contract with National Bureau of Standards for development of Recommendations for National Seismic Design Criteria.

IV. Political Process

<u>November 1974</u>

Concerted effort by Drs. Simpson and Zaldastani and Mr. Remmer to adapt the recommendations of the Committee to appropriate code language.

Dr. Zaldastani carried out an editorial reordering of UBC material to separate all the force requirements in one section and all the "ductility" requirements into another. This work was a major contribution to the final development of the recommended seismic provisions.

<u>December 1974</u>

Letter to Mr. Don Stull, Chairman of the Massachusetts State Building Code Commission, ASCE/BSCE Committee on Seismic Design Criteria.

The recommendations reflect the results of studies of the probable intensities and frequencies of occurrence of future earthquakes in the Commonwealth and represent the Committee's opinion as to the minimum appropriate standards of design and construction for reasonable protection against structural collapse due to a major seismic event. The recommended code provisions can hope to achieve this goal <u>only</u> if construction is carefully inspected by a knowledgeable, competent inspector guided by instructions or inspection specifications prepared by the design engineer. (A strong attempt was made to get inspection requirements.)

<u>December 6 1974</u>

Amendments to the State Building Code - Earthquake Load

filed by the State Building Code Commission with the Secretary of the Commonwealth.

The Commission deleted the section 718.53 Masonry as recommended by the Committee in the hope tha a "more meaningful" section could be developed which would relate to anticipated development of licensing procedures for building supervisors. The inspection indicated by the Committee for masonry was deemed to be unfeasible at the time due to the limited training and experience of inspectors.

January 1, 1975

The Commonwealth of Massachusetts State Building Code including the recommendations of the Seismic Committee were promulgated.

February 1975

The Joint Committee on Seismic Design Criteria arranged two workshops at the Boston Public Library for design professionals, engineers and architects, to explain the content and intentions of the new seismic provisions. Presentation was by Simpson, Zaldastani and Whitman; attendance was about 150.

V. Follow Up

October - November 1975

Earthquake Design Lecture Series

A series of seven lectures was sponsored by the Boston Society of Civil Engineers Section / American Society of Civil Engineers in cooperation with the Massachusetts Institute of Technology, Department of Civil Engineering.

1. Introduction, Fundamentals, Description, Massachusetts Building Code Provisions - K. Tsutsumi
2. Designing Reinforced Concrete Buildings for Resistance to Seismic Effects under the new Massachusetts State Building Code - F. J. Heger
3. Dynamic Analysis of Buildings for Earthquake Design - J.M. Roesset
4. Soils and Foundations - R.V. Whitman
5. Structural Steel - H.J. Degenkolb
6. Masonry -
7. Prefabricated Construction Systems - R.F. Mast

October 1975

Review of Section 718.0 Earthquake Load of the Massachusetts State Building Code

by Daniel M. McGee P.E. Committee on Construction Codes and Standards, American Iron and Steel Institute.

In general it appears the provisions of Section 718.0 of the State Building Code appear to have been adopted from either or both the SEAOC recommendations and the Uniform Building Code. However, some of the modifications or omissions from those generally accepted requirements appear to be incorrect or inappropriate.

The review takes issue with the single Zone 1.5 for the state on the basis of Algermissen map in apparent ignorance of the detailed seismic risk studies on which the Code is based.

Mr. McGee made the following recommendations:

1. Use of a Z value of 3/4 for the eastern portion of Massachusetts and 1/2 for the rest of the state as opposed to the uniform value of 1/3 in the Massachusetts Building Code.
2. Inclusion of an occupancy importance factor as it appears in the 1973 recommendations of the SEAOC Seismology Committee.
3. Updating of the Table 7-3B to include reference by Rock Manufacturers Institute.
4. Exemption of one- and two-family structues from 718.0 should be modified not to include unreinforced masonry buildings.
5. Revision of definition of "dual bracing system" and "space frame".
6. Reference to all appropriate reference standards rather than specifically AISC 1969.
7. A less conservative treatment of steel on the basis of its past performance. AISI feels that the treatment of steel is inconsistent with the treatment of other materials like reinforced and prestressed concrete, which have not performed as well as steel in the past.
8. Weld testing for moment resisting space frames and column splices is seen as overly restrictive for steel.
9. Revision of 718.57 commentary on connections.

(In its overall comments AISI may have been self-serving since more stringent seismic requirements would tend to put concrete at a disadvantage. Detail comments were, however, quite helpful.)

<u>September 29 - October 3, 1975</u>

ICBO Annual meeting where SEAOC recommended lateral force requirements and commentary of 1974 were considered.

(AISI recommended consideration of SEAOC and UBC changes in the formulation of the Massachusetts Code.)

<u>October 30, 1975</u>

Meeting of BSCE/ASCE Committee on Seismic Design Criteria for consideration of proposed revisions to the draft of December 2, 1974.

Suggested revisions and corrections of the text were distributed.

Earthquake Regulation Section of the 1976 Edition of the UBC was distributed.

Prof. Tsutsumi distributed material on background vibration from studies at a Cambridge construction site.

He also distributed data on the comparison of $C = \frac{1}{15\sqrt{T}}$ and $C = \frac{.05}{\sqrt[3]{T}}$ for the calculation of seismic coefficient.

B

APPENDIX B

SECTION 718.0 EARTHQUAKE LOAD

Provisions of section 718 reflect informed judgments regarding the probable intensities of future earthquake ground motions in this region, and their associated probabilities of occurrence. The objective of these provisions is to protect life safety by limiting structural failure.

718.1 GENERAL

a) every building or structure and every portion thereof shall be designed and constructed to resist stresses produced by lateral forces as provided in this section, except detached one and two-family dwellings and minor accessory buildings. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level or to building parts above the foundation. The force shall be assumed to come from any horizontal direction.

b) every building or structure and every portion designed and constructed to resist stresses produced by lateral forces as provided in this section shall be constructed and inspected in accordance with the rules and regulations promulgated by the State Building Code Commission.

718.2 DEFINITIONS: The following definitions apply only to the provisions of this section.

BOX SYSTEM: a structural system where the vertical load is carried by bearing walls and structural framing and where the lateral stability and lateral force resisting system consists of shear walls or braced frames.

BRACED FRAME: a vertical truss or its equivalent which is provided to resist lateral forces in which the members are subjected primarily to axial stresses.

CLASS A SOIL: includes all the classes of soil and rock enumerated in section 723.4.

CLASS A SOIL SITE:

a) a site composed exclusively of Class A soil, or

b) a site where Class A soil overlies or includes Class B soil, provided that the depth below foundation level to the uppermost Class B soil and the cumulative thickness of Class B soil meet the criteria in Figure 7-9.

CLASS B SOIL: includes all classes of soil not qualifying as Class A soil.

CLASS B SOIL SITE: any site which does not meet the criteria for Class A soil site.

DUAL BRACING SYSTEM: consists of a moment resisting space frame and shear walls which meet the following design criteria:

a) the space frame and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and space frame.

b) the shear walls acting independently of the resisting portions of the space frame shall resist the total lateral force.

c) the resisting space frame shall have the capacity to resist not less than twenty-five (25) percent of the total lateral force.

FOUNDATION LEVEL: the lowest of any of the following:

a) the bottom of any spread or combined footing or foundation mat;

b) the bottom of any pile cap;

c) the top of any pier or caisson.

LATERAL FORCE RESISTING SYSTEM: that part of the structural system to which the total lateral forces prescribed in section 718.4 are assigned.

LIQUEFACTION: a term used to describe a group of phenomena occurring in saturated cohesionless sandy and silty soils consisting of a large decrease in effective stress (total stress minus pore pressure) accompanied by large deformations under either static or cyclic loading. The term cyclic mobility should also be included within the scope of the definition of liquefaction.

MOMENT-RESISTING SPACE FRAME: a space frame designed to carry all vertical loads and in which the members and joints are capable of resisting design lateral forces by bending moments.

SHEAR WALL: a wall designed to resist lateral forces parallel to the wall.

SPACE FRAME: a three-dimensional structural system composed of interconnected members, other than bearing walls, designed to function as a complete self-contained laterally stable unit with or without the aid of horizontal diaphragms or floor bracing systems.

718.3 SYMBOLS AND NOTATIONS: The following symbols and notations apply only to the provisions of this section:

C = Numerical coefficient for base shear as specified in section 719.4.

C_p = Numerical coefficient as specified in section 718.4 and as set forth in Table 7-3b.

D = The dimension of the building in feet in a direction parallel to the applied forces.

D_s = The plan dimension of the vertical lateral force resisting system in feet.

$F_i F_n$
F_x = Lateral force applied to level i, n, or x, respectively.

F_p = Lateral force on the part of the structure and in the direction under consideration.

F_t = That portion of V considered concentrated at the top of the structure, at the level n. The remaining portion of the total base shear V shall be distributed over the height of the structure including level n according to Formula (18-5).

$h_i h_n$
h_x = Height in feet above the base to level i, n, or x, respectively.

K = Numerical coefficient as set forth in Table 7-3A.

Level i = Level of the structure referred to by the subscript i.

Level n = That level which is uppermost in the main portion of the structure.

Level x = That level which is under design consideration.

M = Overturning moment at the base of the building or structure.

M_x = The overturning moment at level x.

N = Total number of stories above exterior grade.

T = Fundamental period of vibration of the building or structure in seconds in the direction under consideration.

V = Total lateral load or shear at the base.

$$V = F_t + \sum_{i=1}^{n} F_i$$

where i = 1 designates first level above the base.

W = Total dead load including the partition loading where applicable.

EXCEPTION: W shall be equal to the total deal load plus twenty-five (25) percent of the floor live load in storage and warehouse occupancies; the snow load shall also be included.

W_i / W_x = That portion of W which is located at or is assigend to level i or x respectively.

W_p = The weight of a part or portion of a structure.

γt = Total unit weight.

718.4 MINIMUM EARTHQUAKE FORCES FOR STRUCTURES: The provisions of this section are applicable only to buildings and structures meeting the requirements of section 718.5. All other buildings and structures shall be designed in accordance with section 718.7.

718.41 TOTAL LATERAL FORCE: Every structure shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = 1/3\ KCSW$$

a) C FACTOR

The value of C shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

For all one and two-story buildings or structures the value of C shall be not less than 0.10. For other buildings the maximum value of C need not exceed 0.10.

EXCEPTIONS:

1) C exceeds 0.10 where indicated in Table 7-3b.

2) Buildings or structures which have highly irregular shapes, large differences in lateral resistance or stiffness between different stories or other unusual structural features affecting seismic response shall be designed in accordance with section 718.7.

T is the fundamental period of vibration of the structure in seconds in the direction under consideration. Properly substantiated technical data for establishing the period T may be submitted. In the absence of such data, the value for T for buildings shall be determined by the following formula:

$$T = \frac{0.05h_n}{\sqrt{D}}$$

EXCEPTION: In all buildings in which the lateral force resisting system consists of a moment-resisting space frame which resists one hundred (100) percent of the required lateral forces and which frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces:

$$T = 0.10\ N$$

b) K FACTOR

All buildings shall be designed with a horizontal force factor K = 1 except buildings which have a lateral force resisting system listed in Table 7-3A.

TABLE 7-3A HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]

TYPE OF ARRANGEMENT OF RESISTING ELEMENTS	VALUE OF K
Buildings with a box system as defined in section 718.2	1.33
Buildings with a dual bracing system as defined in section 718.2	0.80
Buildings with a momemt resisting space frame designed to resist the total required lateral force	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building[2]	3.00[3]
Structures other than buildings and other than those set forth in Table 7-3b	2.00

Note 1: Where wind load would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

Note 2: The minimum value of "KC" shall be 0.12 and the maximum value of "KC" need not exceed 0.25.

Note 3: The tower shall be designed for an accidental torsion of five (5) percent as specified in section 718.43. Elevated tanks which are supported by buildings or do not conform to type or arrangement of supporting elements as described

NOTES FOR TABLE 7-3A (continued)

above shall be designed in accordance with section 718.45 using "C_p" = 2.

c) 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, if justified by the results of adequate studies by a qualified registered professional engineer.

718.42 DISTRIBUTION OF LATERAL FORCE

a) VERTICAL DISTRIBUTION

The total lateral force V shall be distributed in the height of the structure in the following manner:

$$F_t = .004V\left(\frac{h_n}{D_s}\right)^2$$

F_t need not exceed 0.15 V and may be considered as 0 for values $\left(\frac{h_n}{D_s}\right)$ of 3 or less, and

$$F_x = \frac{(V - F_t)\ w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

EXCEPTION: One and two-story buildings shall have uniform distribution.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

b) HORIZONTAL DISTRIBUTION

Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm.

718.43 HORIZONTAL TORSIONAL MOMENTS: Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. Where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear-resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five (5) percent of the maximum building dimension at that level.

718.44 OVERTURNING: Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements specified in section 717.0 or the earthquake forces specified in this section, whichever governs.

At any level the incremental changes of the design overturning moment, in the story under consideration, shall be distributed to the various resisting elements in the same proportions as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

718.45 LATERAL FORCE ON PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES: Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = 1/3C_pW_p$$

The values of C_p are set forth in Table 7-3b unless a greater value is required by the basic seismic formula V = 1/3 KCSW. The distribution of these forces shall be according to the gravity loads pertaining thereto.

TABLE 7-3B HORIZONTAL FORCE FACTOR "C" FOR PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over 10 feet in height, masonry or concrete fences over 6 feet in height	Normal to flat surface	0.20
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 6 feet in height plus contents, chimneys, smokestacks and penthouses	Any direction	0.20[1, 2]
When resting on the ground, tank plus effective mass of its contents	Any direction	0.10
Floors and roofs acting as diaphragms[4]	Any direction	0.10
Connections for exterior panels or for elements complying with section 718.64	Any direction	2.00
Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly[5]	Any horizontal direction	0.30

NOTES FOR TABLE 7-3B

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) percent.

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 .16 for the levels below the top two levels.

Note 3: For purposes of determining the lateral force, a minimum ceiling weight of five (5) pounds per square foot shall be used.

Note 4: Floors and roofs acting as diaphragms shall be designed for a minimum value of "C_p" of ten (10) percent applied to loads tributary 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) percent of the floor live load in storage and warehouse occupancies.

be terminated with not less than a ninety (90) degree bend plus a twelve (12) bar diameter extension beyond the boundary reinforcing at vertical and horizontal end faces of wall sections. Wall reinforcement terminating in boundary columns shall be fully anchored into the boundary elements.

2) BRACED FRAMES

a) Reinforced concrete members of braced frames subject primarily to axial stresses shall have transverse reinforcement as specified in 3) below through the full length of the member. Tension members shall additionally meet the requirements for compressive members.

b) In buildings without a moment resisting space frame capable of carrying all vertical loads and the total required lateral force, all members in braced frames shall be designed for 1.25 times the force determined in accordance with section 718.4. Connections for these members are not permitted the thirty-three (33) percent stress increase for earthquake.

3) TRANSVERSE REINFORCEMENT

Where transverse reinforcement is required by the provisions of this section, the amount of such reinforcement shall be not less than that specified below.

The volumetric ratio of spiral reinforcement shall be not less than that specified for reinforced concrete columns, nor less than

$$0.12\ f'_c/f_{yh}$$

Rectangular hoop reinforcement shall be spaced not more than four (4) inches apart and shall have a total cross-sectional area not less than the greater of

$$A_{sh}=0.30\ s_h h_c f'_c/f_{yh}\ (A_g/A_{ch}-1)$$

or

$$A_{sh}=0.12\ s_h h_c f'_c/f_{yh}$$

Single or overlapping hoops may be provided to meet this requirement.

Supplementary cross ties of the same size and spacing as hoops using 135-degree minimum hooks engaging the periphery hoop and secured to a longitudinal bar may be used. Supplementary cross ties or legs of overlapping hoops shall be spaced not more than fourteen (14) inches on center transversely.

718.52 STEEL: Design and construction of earthquake resisting steel framing members and their connections shall conform to the provisions of section 827 and of reference standard AISC 1969 and to the special requirements of this section.

a) MOMENT-RESISTING SPACE FRAMES

1) GENERAL

Design and construction of steel framing in moment-resisting space frames shall conform to the provisions of section 827.0 and the requirements of this section.

2) DEFINITIONS

a) JOINTS: The joint is the entire assemblage at the intersections of the members.

b) CONNECTIONS: The connection consists of only those elements that connect the member to the joint.

3) CONNECTIONS

Each beam or girder moment connection to a column shall be capable of developing in the beam the full plastic capacity of the beam or girder.

EXCEPTION: The connection need not develop the full plastic capacity of the beam or girder if it can be shown that adequately ductile joint displacement is provided with a lesser connection.

4) LOCAL BUCKLING

Members in which hinges will form during inelastic displacement of the frames shall comply with the requirement for "plastic design sections".

5) SLENDERNESS RATIOS

The effective length "kl" used in determining the slenderness ratio of an axially loaded compression member in the moment-resisting space frame depends on its own bending stiffness for the lateral stability of the building, even if bracing or shear walls are provided.

6) NONDESTRUCTIVE WELDING TESTING

Welded connections between primary members of the moment-resisting space frame shall be tested by nondestructive methods for compliance with the Code and job specifications. A program for this testing shall be established by the person responsible for structural design. As a minimum, this program shall include the following:

a) All complete penetration groove welds contained in joints and splices shall be tested one hundred (100) percent either by ultrasonic testing or by radiography.

EXCEPTION: The nondestructive testing rate for an individual welder may be reduced to twenty-five (25) percent subject to the concurrence of the design engineer of record, provided the reject rate is demonstrated to be five (5) percent or less of the welds tested for the welder. A sampling of at least forty (40) completed welds shall be made for such reduction evaluation. Reject rate is defined as the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over three (3) feet in length, each twelve (12) inch increment shall be considered as one weld. For evaluating the reject rate for continuous welds greater than one (1) inch thick, each six (6) inches of length shall be considered one (1) weld.

b) Partial penetration groove welds when used in column splices shall be tested either by ultrasonic testing or radiography as required by the design engineer of record.

b) BRACED FRAMES

1) All members in braced frames of K=1.0 and K=1.33 buildings shall be designed for 1.25 times the force determined in accordance with section 718.4. Connections for these members are not permitted the thirty-three (33) percent stress increase for earthquake.

718.53 MASONRY: Masonry shall be subject to the provisions and reference standards of Article 8.

718.54 TIMBER: Design and construction of earthquake resisting timber structures shall conform to the provisions of section 851 supplemented by the reference standards of Article 8 pertaining to Lumber and Construction and the Timber Construction Manual (second Edition 1974) by the American Institute of Timber Construction, and to the requirements of this section.

a) DIAPHRAGMS

Lumber and plywood diaphragms may be used to resist wind or horizontal earthquake forces.

Design of diaphragms shall conform to the accepted engineering practice as presented in the Timber Construction Manual.

b) Axial and shear forces produced in wood members by wind or earthquake shall be transferred by positive connections and adequate anchorage. Uplift or horizontal displacement of seated connections shall be prevented by positive anchors. Toenailing or nails subject to withdrawal are not acceptable for connections resisting such forces or displacements.

Sheathing materials may be used as tension ties provided the tension force does not provide cross-grain bending or cross-grain tension in the peripheral members or other framing members to which the sheathing connects.

718.55 PREFABRICATED CONSTRUCTION: All structural elements within the structure which are considered to resist seismic forces or movement and/or are connected so as to participate with the structural system shall be designed in accordance with the provisions of this Code in accordance with "Accepted Engineering Practice Standards" (ACI 318-71 for Precast Concrete). Connections shall accommodate all design forces and movement without loss of load carrying capacity of the interconnected members and shall conform to section 718.57.

718.56 OTHER MATERIALS OR METHODS OF CONSTRUCTION: Materials other than concrete, steel, clay masonry, concrete block masonry and wood and structural systems other than structural steel, reinforced concrete, reinforced masonry, wood frame or heavy timber shall not be relied on to resist lateral forces and deformations in building structures unless it can be demonstrated to the building official that the structure can safely withstand lateral distortion eight (8) times that computed for the lateral forces specified in section 718.4. The building official shall require drawings and calculations submitted by a registered professional engineer to verify the requirements of this provision.

718.57 CONNECTIONS

a) Connections with transfer forces between members which resist seismic forces in flexure shall be designed for the required forces and also shall either:

1) Develop the full plastic moment of the member

OR

2) Be capable of deforming to form a reversible plastic hinge.

b) Members which are part of the lateral force resisting system and resist seismic motion by direct axial force shall have connections designed to develop the axial capacities of the members.

c) Connections of structural members, which are not part of the lateral force resisting system, to supporting members shall be designed to resist the required seismic forces without reliance on frictional forces.

d) 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.

e) Connections between diaphragms and resisting shear walls and bracing shall be designed for twice the computed force.

718.6 OTHER DESIGN REQUIREMENTS

718.61 LATERAL FORCE RESISTING SYSTEM: Rigid elements that are assumed not to be part of the lateral force resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design.

718.62 MOMENT RESISTING SPACE FRAMES: Moment resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

718.63 BUILDING SEPARATIONS: All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

718.64 SETBACKS: Buildings having setbacks wherein the plan dimension of the tower in each direction is at least seventy-five (75) percent of the corresponding plan dimension of the lower part may be considered as a uniform building without setbacks for the purpose of determining seismic forces.

For other conditions of setbacks the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

EXCEPTION: Nothing in this subsection shall be deemed to prohibit the submission of properly substantiated technical data for establishing the lateral design forces by a dynamic analysis in accordance with section 718.7

718.65 COMBINED VERTICAL AND HORIZONTAL FORCES: In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

718.66 EXTERIOR ELEMENTS: Precast, nonbearing, non-shear wall panels, parapets, or other elements which are attached to, or enclose the exterior, shall accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of poured-in-place concrete or by mechanical fasteners in accordance with the following provisions:

a) 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.

b) Connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

c) Connections to permit movement in the plane of the panel for story drift may be properly designed sliding connections using slotted or oversize holes or may be connections which permit movement by bending of steel.

718.67 MINOR ALTERATIONS: Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this section of the Code.

718.68 DRIFT: Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted practice. Lateral deflection of diaphragms shall be considered in addition to the deflection of vertical bracing elements.

Rigid elements that are assumed not to be part of the lateral force resisting system may be incorporated into buildings provided that the effect of the action of the system is considered and provided for in the design. In addition, the effects of the drift on such rigid elements themselves and on their attachment to the building structure shall be considered.

718.69 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) 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 underlain at shallow depths by cohesionless granular soils, the blow counts of which only slightly exceed the criteria given in Figure 7-10, adequate consideration shall be given to the lateral and vertical movements of footings that may occur during the design earthquake specified in section 718.7.

718.70 RETAINING WALLS: Retaining walls shall be designed to resist at least the superimposed effects of the total static lateral soil pressure, excluding the pressure caused by any temporary surcharge, plus an earthquake force of $0.045\gamma_t H^2$ (Horizontal backfill surface). Surcharges which are applied over extended periods of time shall be included in the total static lateral soil pressure and their earthquake lateral force shall be computed and added to the force of $0.045\gamma_t H^2$. The earthquake force from the backfill shall be distributed as an inverse triangle over the height of the wall. The point of application of the earthquake force from an extended duration surcharge shall be determined on an individual case basis. If the backfill consists of loose saturated granular soil, consideration shall be given to the potential liquefaction of the backfill during the seismic loading.

718.71 DYNAMIC ANALYSIS: Any building or structure is deemed to have complied with the provisions of section 718 if a qualified registered engineer determines that there is negligible risk to life safety if the building or structure experiences an earthquake with a peak acceleration of 0.12g and a frequency content similar to that implied by the appropriate response spectrum in Figure 7-10. A copy of the studies upon which the determination may be based upon shall be filed with the building official. Such a determination may be based upon

a) a dynamic analysis, based upon generally acceptable procedures, together with evidence that the building or structure can safely withstand the computed displacements and distortions;

b) a comparison of the building or structure with similar buildings or structures having similar foundations and subsoil conditions, that have withstood a similar actual earthquake; or

c) other accepted procedures.

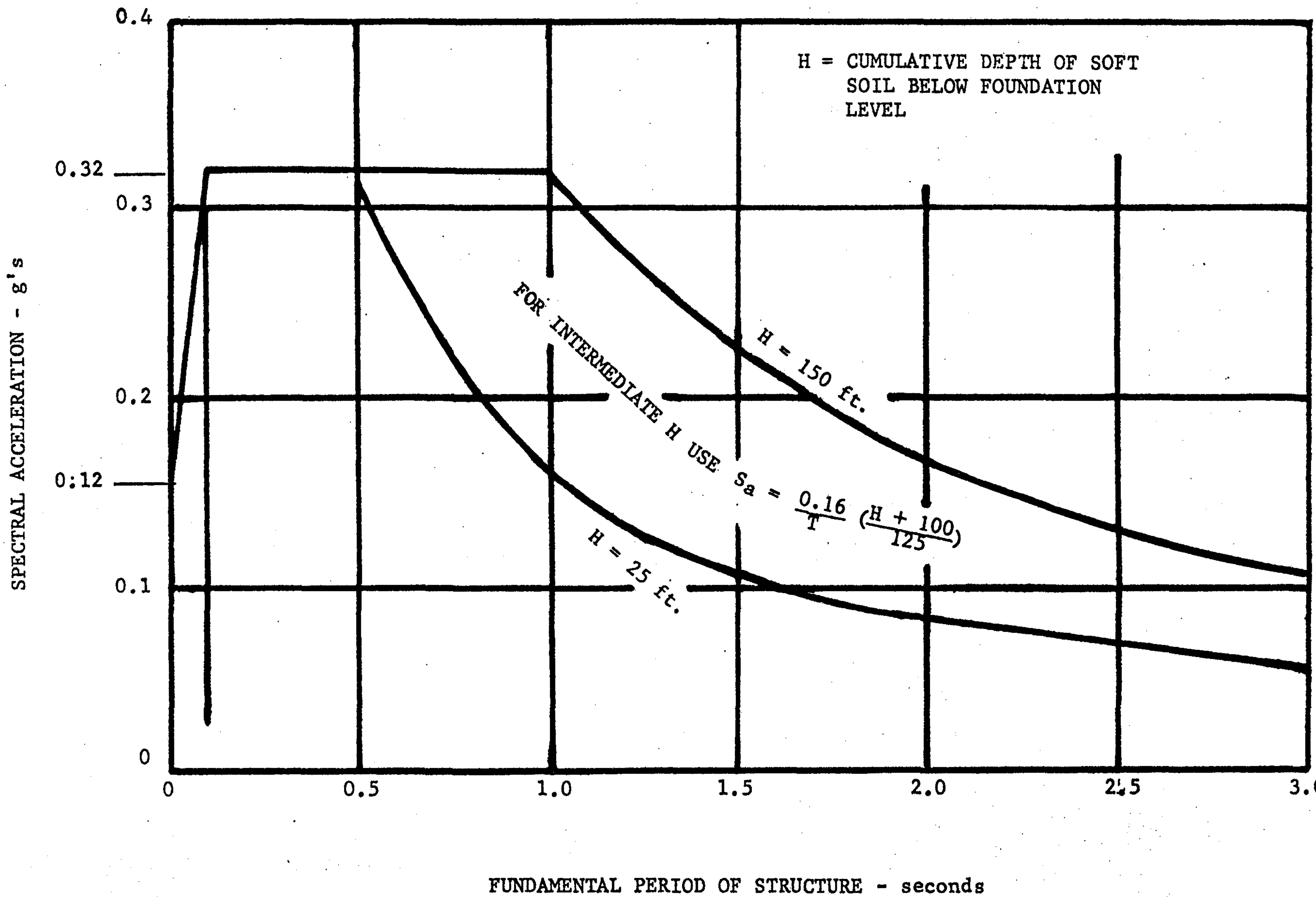
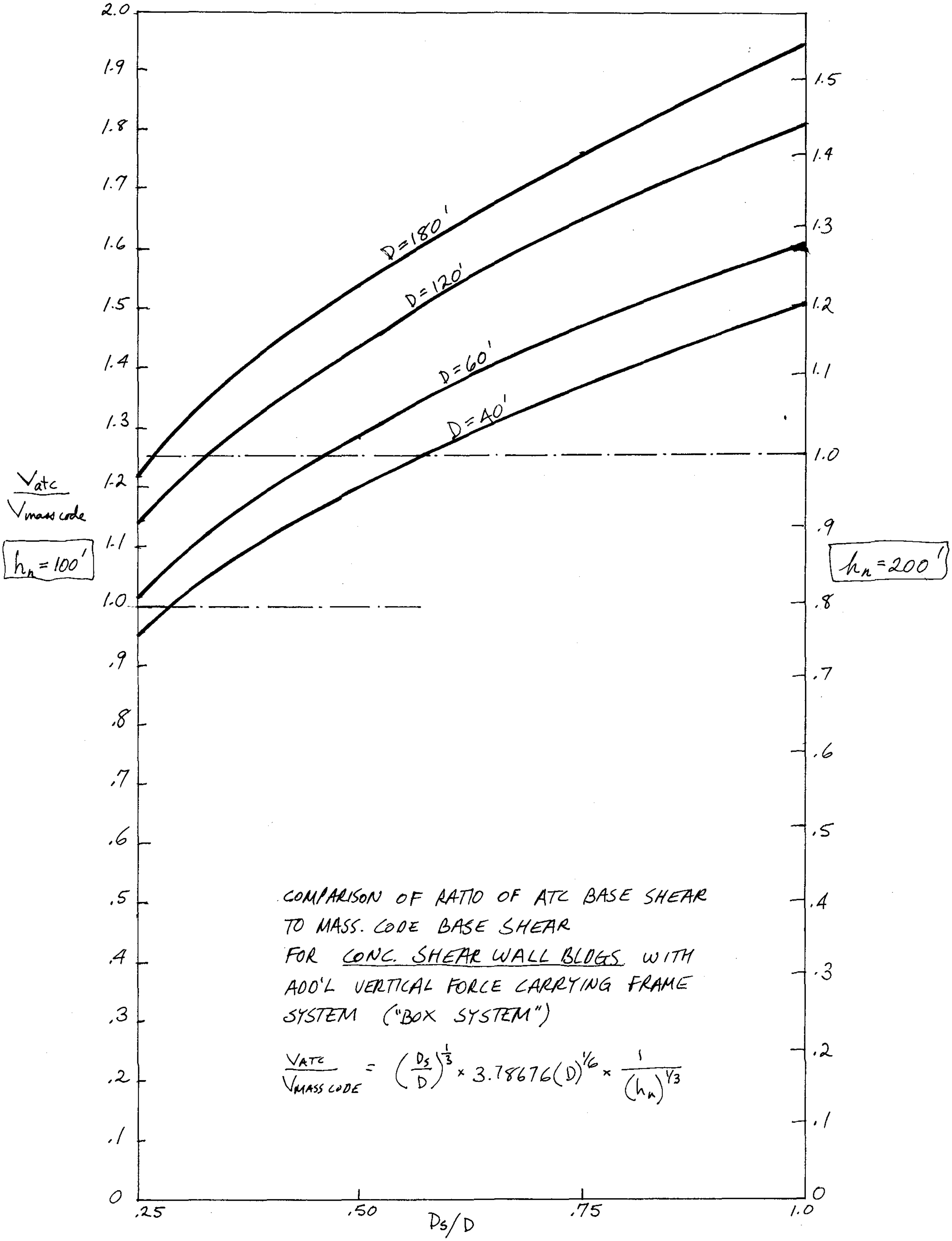


Figure 7-10 DESIGN RESPONSE SPECTRUM

C

APPENDIX C


2.0
1.9
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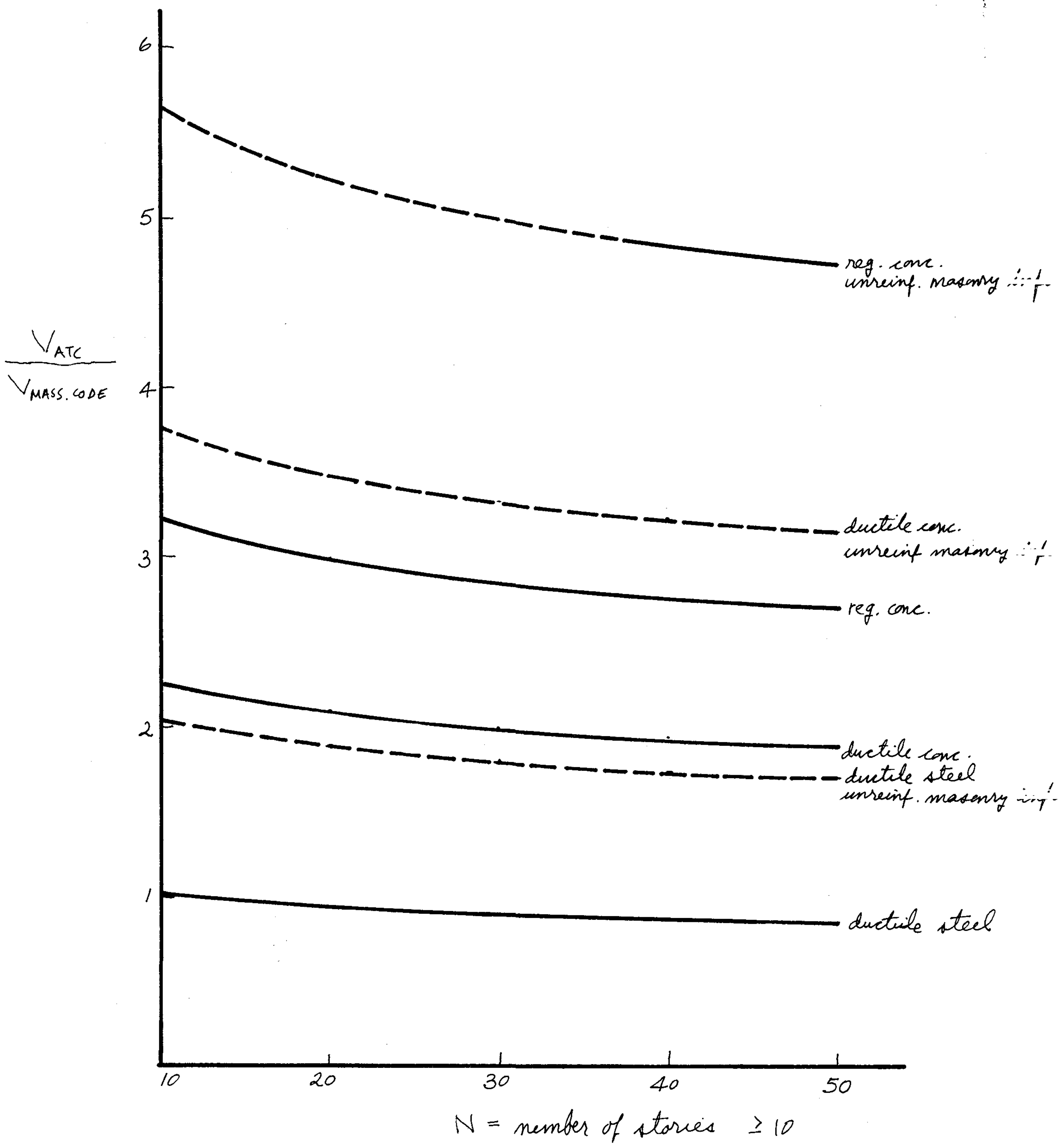
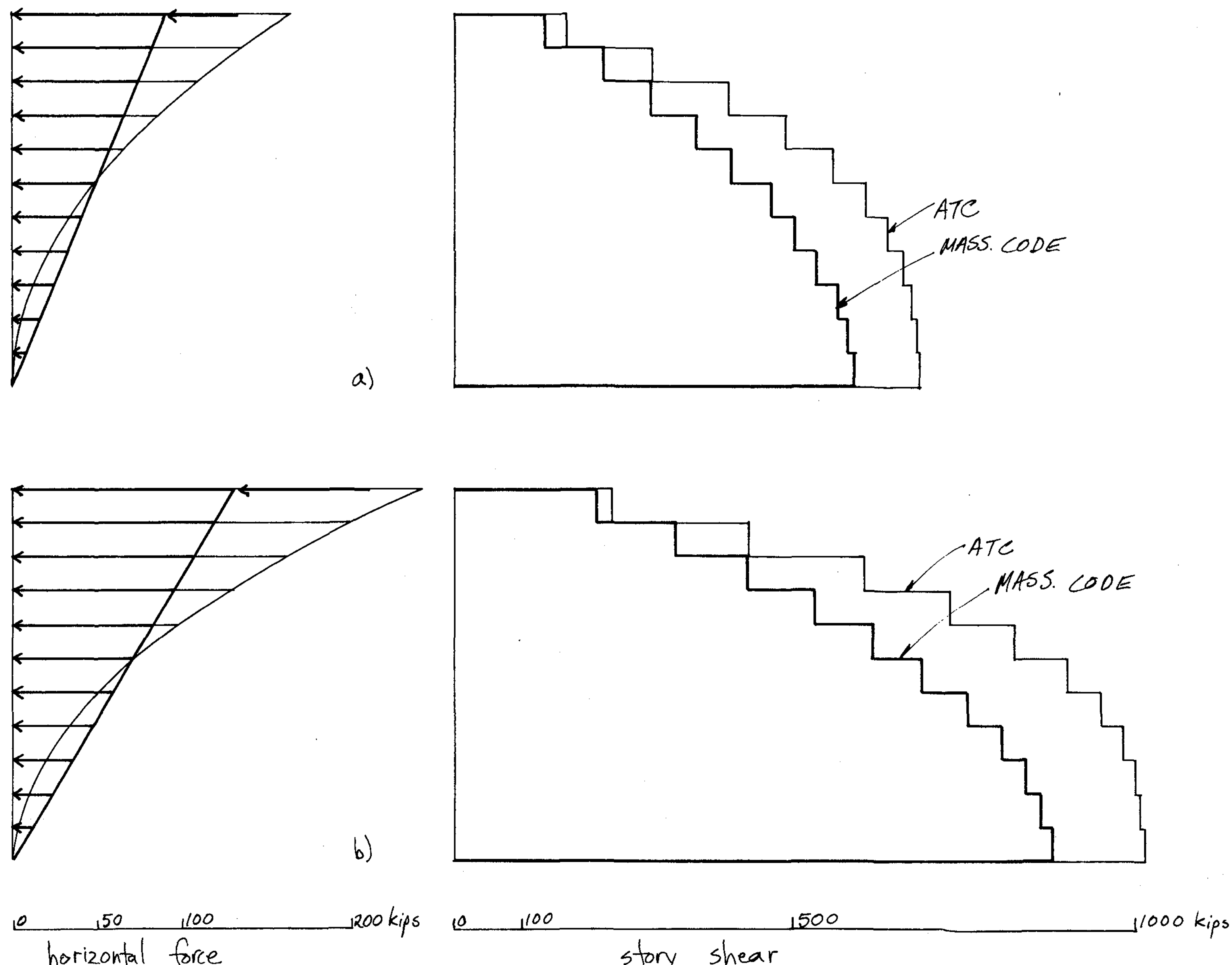
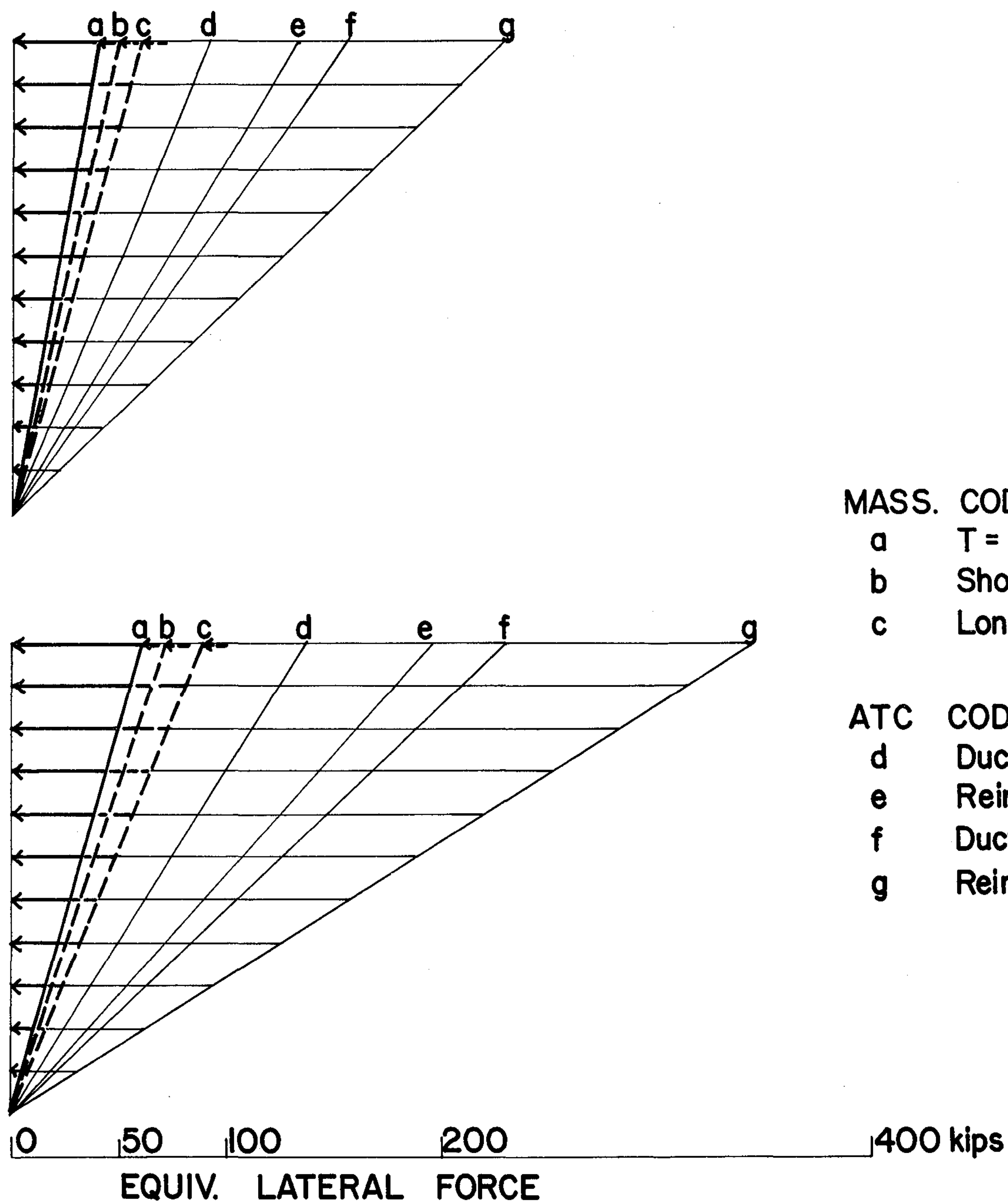
0
$\frac{V_{atc}}{V_{mass\ code}}$
$h_n = 100'$
$h_n = 200'$
D=180'
D=120'
D=60'
D=40'
.25
.50
.75
1.0
Ds/D
COMPARISON OF RATIO OF ATC BASE SHEAR
TO MASS. CODE BASE SHEAR
FOR CONC. SHEAR WALL BLDGS. WITH
ADD'L VERTICAL FORCE CARRYING FRAME
SYSTEM ("BOX SYSTEM")
$\frac{V_{ATC}}{V_{MASS\ CODE}} = \left(\frac{D_s}{D}\right)^{\frac{1}{3}} \times 3.78676(D)^{1/6} \times \frac{1}{(h_n)^{1/3}}$

COMPARISON OF RATIO OF ATC BASE SHEAR
TO MASS. CODE BASE SHEAR
FOR (UNRESTRICTED) <u>MOMENT RESISTING FRAME BLDGS</u> ≥ 10 stories





11 story conc. shear wall, short direction a) good soil, b) bad soil



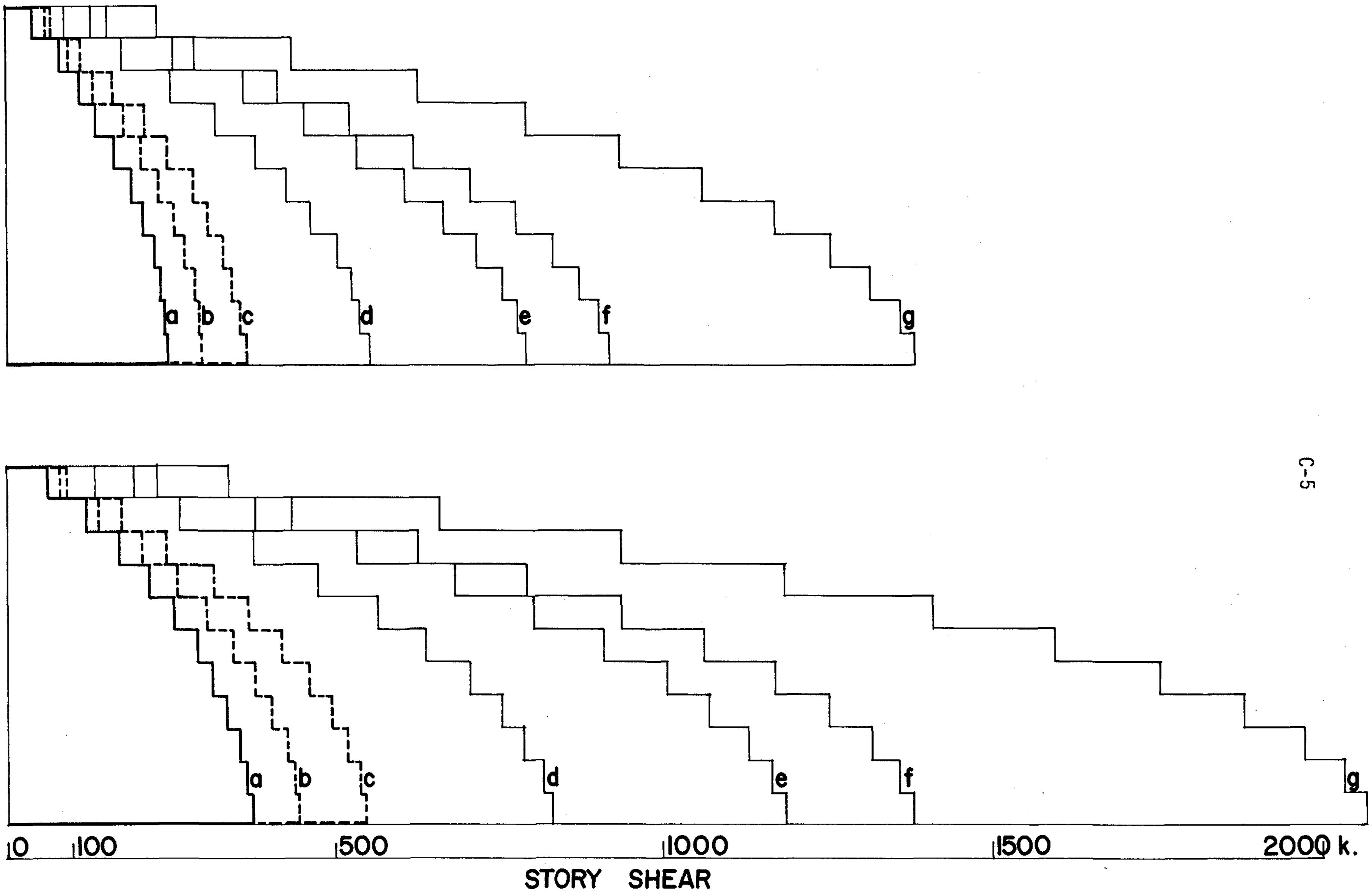
MASS. CODE

- a T = 0.1N
- b Short direction $T = 0.05\,h_n/\sqrt{D}$
- c Long direction $T = 0.05\,h_n/\sqrt{D}$

ATC CODE

- d Ductile reinforced concrete
- e Reinforced concrete
- f Ductile r/c, unreinf. masonry infill
- g Reinf. conc., unreinf. masonry infill

11 STORY CONC. MOMENT RESISTING FRAME TOP - GOOD SOIL BOTTOM - POOR SOIL



II STORY CONC. MOMENT RESISTING FRAME TOP - GOOD SOIL BOTTOM - POOR SOIL

C-6

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3. Anagnostopoulos, "Nonlinear Dynamic Response and Ductility Requirements of Building Structures Subject to Earthquakes", Dept. of C.E. R72-54, ST No. 349, September 1972.

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12. Isbell and Biggs, "Inelastic Design of Building Frames to Resist Earthquakes", Dept. of C.E. R74-36, ST No. 393, May 1974(NTIS PB233-856/AS)

13. Ackroyd and Biggs, "The Formulation and Experimental Verification of Multistory Buildings", Dept. of C.E. R74-37, ST No. 394, May 1974(NTIS PB#237762/

14. Taleb-Agha, "Sensitivity Analyses and Graphical Methods for Preliminary Solutions", Dept. of C.E. R74-41, ST No. 403, June 1974.(NTIS PB#237-187/AS)

15. Panoussis, "Seismic Reliability of Infinite Networks", Dept. of C.E. R74-57, ST No. 401, September 1974.(NTIS PB#238744/AS)

16. Whitman, Yegian, Christian, and Tezian, "Ground Motion Amplification Studies, Bursa, Turkey", Dept. of C.E. R74-58, ST No. 402, September 1974. (NTIS PB#)also pub. Earthquake Engr. J., John Wiley & Sons, Jan. 19[illegible]

17. de Neufville, "How do we Evaluate and Choose Between Alternative Codes for Design and Performance", Dept of C.E. R75-3, No. 510.NTIS PB#259184/AS)

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19. Wong, "Correlation Between Earthquake Damage and Strong Ground Motion", Dept. of C.E. R75-24, Order No. 505, May 1975. (NTIS PB# 259186/AS)

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