NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo

RECOMMENDED MODIFICATIONS TO ATC-14

by

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Technical Report NCEER-89-0012

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NOTICE

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C.D. Poland[1] and J.O. Malley[2]

Project Review Panel

Dr. Peter Gergely - Cornell University
Dr. Richard White - Cornell University
Mr. Glen R. Bell - Simpson, Gumpertz and Heger
Mr. Warner Howe - Gardner and Howe
Dr. Charles Lindbergh - The Citadel and Lindbergh and Associates

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1 Principal-in-Charge, H.J. Degenkolb Associates, Engineers
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PREFACE

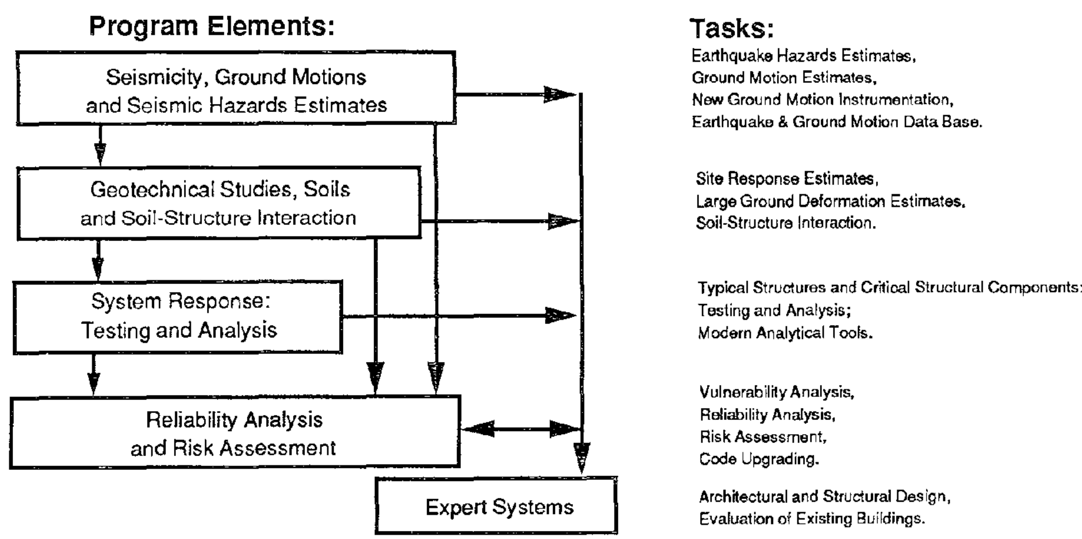
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to reliability analysis and risk assessment.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. This work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



Reliability analysis and risk assessment research constitutes one of the important areas of Existing and New Structures. Current research addresses, among others, the following issues:

1. Code issues - Development of a probabilistic procedure to determine load and resistance factors. Load Resistance Factor Design (LRFD) includes the investigation of wind vs. seismic issues, and of estimating design seismic loads for areas of moderate to high seismicity.
2. Response modification factors - Evaluation of RMFs for buildings and bridges which combine the effect of shear and bending.
3. Seismic damage - Development of damage estimation procedures which include a global and local damage index, and damage control by design; and development of computer codes for identification of the degree of building damage and automated damage-based design procedures.
4. Seismic reliability analysis of building structures - Development of procedures to evaluate the seismic safety of buildings which includes limit states corresponding to serviceability and collapse.
5. Retrofit procedures and restoration strategies.
6. Risk assessment and societal impact.

Research projects concerned with reliability analysis and risk assessment are carried out to provide practical tools for engineers to assess seismic risk to structures for the ultimate purpose of mitigating societal impact.

Research on the safety assessment of existing buildings ranges from engineering-type broad evaluation methods to detailed experimental and analytical studies of specific types of common buildings or components. This report belongs to the former category and addresses many important questions in safety evaluation of buildings which are specific to the midwest and east in the United States. Numerous additions and alterations to the ATC-14 document, which emphasizes problems in more active seismic regions, are summarized in this report. It is expected that NCEER will also publish a complete ATC-14 type document valid for regions of moderate seismicity which will include all the modifications reported here.

ABSTRACT

The recently released ATC-14 document "Evaluating the Seismic Resistance of Existing Buildings" has been proposed as a viable methodology for the seismic evaluation of existing buildings. The ATC-14 document was written to apply nationwide, although the emphasis was placed on buildings in regions of high seismicity. As a portion of the "Existing Structures" topic, one of the three major areas of research for the second year of the National Center for Earthquake Engineering Research's (NCEER) five-year program, an investigation was performed to critically assess the applicability of ATC-14 to buildings in regions of low seismicity, such as the Eastern United States. H.J. Degenkolb Associates, the Subcontractor who developed ATC-14 for the Applied Technology Council, was contracted to direct this assessment. They organized a panel of five engineers from the Eastern United States experienced in seismic design and evaluation to review the document. A two-day project meeting was held to discuss the review comments of the panel and to determine areas where ATC-14 could be improved. A number of major areas of potential improvement were identified and developed for inclusion in future editions of ATC-14. These improvements, which are presented in this report, include the following:

1. A discussion of other NCEER projects which are studying topics which could provide results that would be useful to future editions of ATC-14. Future research topics which could improve ATC-14 are suggested.

2. A discussion of the present state of knowledge on Eastern United States seismicity which occurred during a meeting with seismologists in conjunction with an NCEER sponsored conference on eastern earthquake hazards.

3. A description of the regional similarities and differences which exist between the Eastern and Western United States in seismic design and evaluation.

4. A collection of additional information which could be useful in a seismic evaluation. This information includes a list of historical documents on building construction, an expanded list of reference standards, a compilation of state code adoption status and a list of earthquake damage data for Eastern United States earthquakes.

5. A major revision and expansion of the ATC-14 sections which provided the seismic evaluation procedure for buildings in regions of low seismicity.

6. A major revision and expansion of the ATC-14 Chapter on non-structural elements.

The information presented in this document should serve as an excellent supplement to ATC-14 and will be especially useful for the seismic evaluation of buildings in regions of low seismicity.

Because of the volume of the recommended modifications, NCEER has decided to fund a follow-up project which will incorporate all of the information presented here with the original ATC-14 document to generate a new document which is specifically intended for the seismic evaluation of buildings in regions of low seismicity. This report will provide a valuable tool for engineers performing these evaluations on buildings in the Eastern United States.

ACKNOWLEDGMENT

This project was funded by Grant 87-0001 from the National Center for Earthquake Engineering Research, for which the authors are grateful. The project review panel, which consisted of Peter Gergely, Dick White, Glen Bell, Charles Lindbergh, and Warner Howe, provided a great deal of effort which is sincerely appreciated. The authors would also like to thank the Applied Technology Council for providing disks which contained the ATC-14 text which were used in developing the final report. The authors also appreciate the efforts of Dianne Quirley and Chimene Camacho who typed the manuscript.

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CHAPTER 1

INTRODUCTION

Modern building codes (hereafter referred to as the code), such as the NEHRP Provisions, are written to guide the construction of new buildings. They hopefully gather up all of the available collective experience in the behavior of structures and present it in a way that is applicable to all forms of construction. Within the seismic provisions, building code standards are developed with life safety, damage control, and cost in mind. Hopefully, the result is a complete building system that costs slightly more to build and has the proper strength and connection details necessary to successfully resist earthquake forces. Experience has shown that this successful performance depends on the base shear strength of the main structural elements, the strength and ductility of the connections, the building configuration, the material type, and the interconnection of the structural parts.

Unlike the traditional structural design for dead and live loads, seismic design anticipates that the buildings will be damaged after a truly major event. To design buildings to be damage-free would not only be very expensive but would also severely limit the permissible styles of construction. New buildings are generally designed to be strong enough to resist small earthquakes without damage and major earthquakes without collapse. To accomplish this goal, the structural design based on the code involves a combination of basic lateral force resisting strength, with a proper structural configuration, and appropriate interconnection of the structural elements. In fact, within the code, there is a direct relationship between how a building is configured, detailed, and tied together and the amount of lateral force for which it is designed. This interrelationship does not exist in most existing buildings. For that reason, the code is not a suitable standard for their evaluation.

A proper detailed seismic evaluation of a building needs to focus on the "weak links" of the structure which have been shown to be critical in past earthquakes in order to assess their susceptibility to catastrophic damage. If the level of expected damage is determined to be unacceptable, then these "weak links" need to be strengthened and/or new seismic resistant systems installed.

With funding provided by the National Science Foundation, the Applied Technology Council (ATC) has developed and published a methodology, ATC-14, for evaluating specific buildings that is tailored for use by practicing structural engineers. This methodology leads not only to conclusions concerning the adequacy of the structure for a given event, but also identifies the structure's weaknesses and, therefore, areas of needed rehabilitation. It has been structured to permit the rapid screening of a large inventory of buildings followed by detailed evaluation where necessary.

ATC-14 was developed to be consistent with the latest building codes, but tailored to the often non-conforming characteristics of the variety of buildings in existence. It was specifically aimed at assessing a building's life safety level of resistance, with a recommendation that all buildings be strengthened to this minimum level.

Life safety in this work was defined broadly as damage that would likely kill an occupant, cause injury to the point of immobility or block any of the dedicated means of egress from the building. The process also identifies areas of potential damage, including "non-structural" elements, but stops short of determining actual expected damage levels. It was developed for application throughout the United States.

It is important to note that ATC-14 has set an evaluation standard that is less stringent than modern building codes. It is applicable only to existing buildings and anticipates that in the worst case a building meeting its requirements may be severely damaged and perhaps irreparable after a major earthquake. The building will have hopefully provided a safe refuge for its occupants during the event. This level of performance is not acceptable for new construction because superior earthquake performance can be accomplished through proper design at little increase to construction costs.

ATC-14 was based on a lengthy review of the available literature, a State of Practice review, seismic design provisions currently in use [1,2,3,4] and lengthy discussions between the Subcontractor and the Project Engineering Panel regarding all aspects of the project. It was published as a complete document that includes the actual methodology, all background material and four examples. It was written not only as a working handbook but also as an educational tool. The persistent reader is also rewarded with an overview of the State of Practice in this field, a discussion of ground motion criteria, a detailed description of structural behavior in past earthquakes and an extensive list of references and related material.

1.1 Impetus for this Project

Awareness of the seismic hazard which exists in the Central and Eastern United States is expanding rapidly. The potential for damaging earthquakes in these regions of the United States is becoming better understood. The most recent editions in the building codes which are used for the construction of new buildings in the Eastern United States now include mandatory seismic design provisions. These advances will undoubtedly increase the seismic resistance of new buildings constructed in these areas of the country.

But, the vast majority of the buildings in the United States were constructed without the benefit of specific design for seismic forces. These buildings will have some inherent seismic resistance from their capacity to withstand wind forces. But, most of the detailing and strength requirements prescribed by modern seismic codes were not included in typical construction practice. Existing buildings in the Central and Eastern United States, therefore, constitute a serious threat to life safety in the event of a major earthquake.

ATC-14 was intended to be applicable to buildings throughout the United States. Buildings of high and moderate seismicity are addressed together in the same sections. It includes separate procedures for evaluating buildings in regions of low seismicity ($A_A \leq .10g$). This separation was a deliberate attempt to recognize the variations of seismic hazard, design practice, and other factors between the regions of low and high seismicity.

But, none of the individuals who directly participated in the development of the document were practicing engineers from the Eastern United States. Because of this lack of input, it was felt that there may be portions of the document which do not adequately address the construction practices typically employed in regions of low seismicity. It was therefore determined that a critical review of the ATC-14 document was required to assess its applicability to buildings in the Eastern United States.

1.2 Overview of this Project

The National Center for Earthquake Engineering Research (NCEER) funded this critical review of ATC-14 for its applicability to the Eastern United States. This review is one of many projects which NCEER sponsored to study the seismic response of existing structures. This broad topic was one of the major areas of study for the second year of the five-year plan being coordinated by NCEER. Other projects being performed as part of the existing structures topic include experimental research, analytical studies, and reliability assessments. NCEER is coordinating and integrating the results of these various projects.

H.J. Degenkolb Associates of San Francisco, California, the Subcontractor and primary author of the ATC-14 document, served as the Principal Investigator for this project. A review panel of five engineers from the Eastern United States who are knowledgeable in seismic design were selected to participate in this project. Table 1.1 lists the project participants.

In the first phase of the project, each member of the review panel performed an in-depth critical review of the applicability of the ATC-14 procedure to typical construction in the Eastern United States. A two-day meeting was then held in Memphis, Tennessee to discuss the results of these reviews. This discussion developed a list of issues which the review panel identified as areas where the original ATC-14 document could be improved or expanded in order to be more applicable to Eastern construction. These issues were arranged into a number of work items which were then assigned to various members of the project team, who would then develop the information needed to propose modifications and/or additions to the ATC-14 document.

After collecting all of the information developed by the project members, Degenkolb Associates convened another two-day project meeting. This meeting, which occurred in Boston, Massachusetts, included a discussion of the proposed revisions and modifications, the assignment of a few final work items, and the planning of the structure of the project report.

Appendix A includes the Meeting Minutes developed during the course of this project.

1.3 Format of the Report

This report is oriented around the major suggestions for modification and addition to the ATC-14 document which were developed by the project team. These topics, which each compose a chapter of this report, include the following:

- Liaison with Other NCEER Projects
- Seismicity Issues
- Regional Differences Between the Eastern and Western United States in Earthquake Engineering
- Additional Information Which could be of Use to Evaluating Engineers
- A Complete Revision of the Sections on the Seismic Evaluation Buildings in Regions of Low Seismicity
- A Major Revision to the Chapter on Non-Structural Elements

1.4 Other Projects Related to this Work

As the size of this report demonstrates, this project has generated a large volume of recommended modifications to ATC-14 for the seismic evaluation of buildings in areas of low seismicity. Since this report only chronicles suggested improvements to ATC-14, an engineer reviewing a building in a region of low seismicity would be required to use both documents to incorporate all of the best information available to guide the seismic evaluation. This requirement would make performing the evaluation difficult and cumbersome for the structural engineer. This undesirable situation would undoubtedly result in a reduced application of either of the documents for the seismic evaluation of buildings in regions of lower seismicity.

In order to avoid this undesirable situation, it was felt that it would be beneficial to incorporate all of the suggested revisions generated by the present NCEER project into ATC-14 to develop a single document for the seismic evaluation of existing buildings in regions of low seismicity. Such a document would be of great benefit to structural engineers performing such evaluations in the Eastern United States. As a result, NCEER has decided to fund a follow-up project to incorporate the information generated in this project to develop a document for the seismic evaluation of existing buildings which is specifically intended for structures in regions of low seismicity. This document, which should be published in mid-1989, will become a valuable tool for engineers performing seismic evaluations on buildings in the Eastern United States.

A number of other projects are presently underway which directly relate to this work. One is ATC-22, which is a FEMA (Federal Emergency Management Association) sponsored project to develop a handbook for the seismic evaluation of existing buildings. This project is using ATC-14 as the basis of the handbook procedure. FEMA is also sponsoring a companion document which will outline appropriate strengthening procedures to mitigate potential hazards identified in the seismic evaluation. The recommendations generated by this NCEER project will be available for incorporation by ATC-22 and the other FEMA document, as well as for any future revised editions of ATC-14. The information presented in this report should serve as an excellent supplement to ATC-14, and will be especially useful for the seismic evaluation of buildings in the Eastern United States.

TABLE 1.1

LIST OF PROJECT PARTICIPANTS

Review Panel Members

Peter Gergely	-	Cornell University, Ithaca, New York
Richard White	-	Cornell University, Ithaca, New York
Glen Bell	-	Simpson, Gumpertz, and Heger, Arlington, Massachusetts
Warner Howe	-	Gardner and Howe, Memphis, Tennessee
Charles Lindbergh	-	The Citadel and Lindbergh and Associates, Charleston, South Carolina

Principal Investigator

H.J. Degenkolb Associates, Engineers, San Francisco, California

Chris D. Poland	-	Principal-In-Charge
James O. Malley	-	Structural Engineer

CHAPTER 2

INTERACTION WITH OTHER NCEER PROJECTS

During the development of the ATC-14 document, it became apparent that a number of the issues of concern which were to be included in the seismic evaluation procedure were not adequately understood. Because of this lack of understanding, the analysis procedures recommended for the detailed evaluation of these issues are often general techniques to address the topic under consideration. While these general techniques can provide the evaluator with basic information on the adequacy of the condition being reviewed, in many cases, it was felt that more detailed and/or appropriate procedures would result in more accurate conclusions. The development of these more accurate analysis techniques would definitely improve the evaluation procedure.

One of the primary reasons for establishing the National Center for Earthquake Engineering Research (NCEER) was to foster and encourage cooperation and integration between investigators in the field of earthquake engineering research. This project could significantly benefit from this cooperative research by utilizing the results from other projects which are addressing topics where the ATC-14 project lacked the information necessary to make prescriptive recommendations. In addition, this project could assist in the continued research to be performed by NCEER investigators through identifying other areas where information is lacking but no present studies are planned.

This Chapter will discuss the areas of interaction between this project and other research being performed by NCEER investigators. This will include a discussion of the current NCEER projects which could provide information that would improve the ATC-14 document. The second section will present a list of possible topics for future research which were identified as the most critical areas for filling the present gaps of knowledge identified during the development and review of ATC-14.

2.1 Other NCEER-Sponsored Projects Which Could Improve ATC-14

There are a number of other NCEER sponsored projects which could provide information that would improve the evaluation procedure presented in ATC-14. This section will discuss the following items concerning these projects:

1. The content of the project, names of the investigators and location of the work.
2. The specific area(s) of the ATC-14 document which could benefit from the results of this project.
3. The status of the project and expected date of completion.

2.1.1 Projects Investigating Lightly Reinforced Concrete

A cooperative effort between researchers at Cornell, Lehigh, Rice and SUNY at Buffalo is underway to investigate the behavior of concrete structures and components which have been identified as being especially susceptible to damage under seismic loading. This effort includes both analytical and experimental investigations of common building details for concrete structures designed without consideration of lateral loads. Prototype buildings of three, six and nine stories have been designed using typical practice for structures in regions of low seismicity. These prototype structures are being studied using a number of available analysis programs. These analyses will assist the investigators in planning and evaluating the subsequent experimental work. A series of small scale tests are planned for testing on the Cornell Shaking Table, including a complete two-story building which was designed to correlate with a larger scale test at U.C. Berkeley. Other medium scale shaking table tests will be performed at SUNY at Buffalo. A series of diaphragm tests are planned by the investigators at Lehigh. A number of full size tests will be performed at Cornell on beam-column joint subassemblages with lightly confined column bar splices above the floor level and discontinuous longitudinal beam reinforcement. Research at Rice University is addressing flat-plate construction and lightly reinforced concrete elements. The principal investigators for these projects are

Professors Gergely and White at Cornell, Huang and Lu at Lehigh, Durrani at Rice, and Ketter and Reinhorn at SUNY at Buffalo.

The results of these projects could provide a body of information which could improve the evaluation procedures for all types of concrete construction (Chapters 7 and 8 of ATC-14). Especially useful information may be gained from the results of the beam-column joint tests on specimens without ductile detailing. The results of these tests could be used to update the ATC-14 statements and procedures for the detailing requirements of concrete frame buildings, such as Statements 7.1.6.13 through 7.1.6.20.

As of August 1988, some of the initial experimental work has been performed on these projects, so some preliminary information will be available shortly. More definitive information from these projects will be available in 1989.

2.1.2 Projects Investigating Semi-Rigid Connections in Steel Framed Buildings

Three NCEER sponsored projects are currently performing experimental and analytical studies on semi-rigid connections in steel framed buildings. At the University of South Carolina, Professors Radziminski, Dickerson and Bradburn are performing tests on connections with top and seat angles, and double web angles. Similar studies are being performed at SUNY at Buffalo by Professor Reinhorn, except that the test specimen will only have top and seat angle connections. Professors Leon and Galambos at the University of Minnesota are investigating contribution of composite action between the floor slabs and the semi-rigid steel frames.

Steel framed buildings with semi-rigid connections were a very common form of construction in California before the 1940s, and are still widely used in areas of low seismicity such as the Eastern United States. The information from these projects would be useful in developing analysis procedures appropriate for evaluating frames with semi-rigid connections. These procedures would be added to Statement 6.1.6.9.

The majority of the tests for these three projects should be complete by the end of Summer of 1988, with the reports completed by the end of the year.

2.1.3 Projects on the Development of Expert Systems

Three institutions, Cornell, Carnegie-Mellon, and Lehigh are participating in projects to develop expert systems for use in Earthquake Engineering.

The Cornell and Lehigh projects are developing a knowledge-based expert system that is intended to assist practicing engineers in the seismic design of new buildings. This work is incorporating the opinions obtained from questionnaires which were sent out to a number of experts in this field. Probabilistic methods may also be incorporated into this system. The Carnegie-Mellon project is developing an expert system for the seismic evaluation of existing buildings. This project is using the ATC-14 document as the basis for the expert system for evaluating existing buildings.

The following individuals are serving as principal investigators for the Expert Systems projects: Professors Gergely and Abel at Cornell, Professors Wilson and Mueller at Lehigh, and Professors Fenves and Bielak at Carnegie-Mellon.

The modifications to the ATC-14 document which are recommended by this study will be of use to the Carnegie-Mellon project which is basing the expert system for seismic evaluations on the information in ATC-14. This Carnegie-Mellon project is planning to incorporate the proposed revisions into their expert system.

At present, the two expert systems are in the development stage. These projects held a Workshop in August 1988 in which a number of experts tested the expert systems and provided suggestions for areas where improvements could be made. The expert system for seismic evaluations had developed a procedure for one model building type for this Workshop. Since the expert systems projects are in the developmental stages, the recommendations of this project can be easily incorporated into a system for seismic evaluations.

2.1.4 Project on Seismic Evaluation of Buildings in New York City

Weidlinger Associates is performing an evaluation of the earthquake resistance of high-rise residential flat slab concrete buildings typically constructed in New York City. They used two actual buildings as the basis for the study. Lateral analyses for both wind and seismic forces were performed. The wind loads were based on the BOCA and NYC Codes. Three input seismic response spectra were used: ATC-3 spectra, ATC-14 spectra and a local site dependent spectra for the actual site. Three-dimensional linear analyses were performed to determine the story shear and overturning demands. These demands were then compared to the capacities calculated using the ACI Code.

The results of this evaluation will be of interest to this project since it will provide a quantitative analysis of the ATC-14 spectra as it relates to wind and other seismic spectra. These analyses can then be compared with qualitative impressions of these structures as a check to verify the appropriateness of the ATC-14 response spectra.

Preliminary results of the linear analyses are presently available. Future analyses of these two buildings will incorporate material nonlinear effects. The report for this project should be available for review by the end of 1988.

2.1.5 Projects on Ground Motion

A number of projects on ground motion are presently being performed by investigators at Lamont-Doherty, Rensselaer, and SUNY Buffalo. These projects include a project led by Dr. Jacob which will develop a set of synthetic seismograms for ground motion characteristic of Eastern North America and a comprehensive data base for ground motion and seismic hazards. Also at Lamont-Doherty, Drs. Seeber and Tuttle are collecting macroseismic and instrumental data on seismic sources in the Eastern United States. Professor Papageorgiou at Rennselaer is studying a recent mid-plate earthquake in an effort to simulate the motion for a large New Madrid earthquake. Professors Budhu and Giese at SUNY at Buffalo are studying the liquefaction potential of the soils of the Eastern United States.

As these projects develop, information will be generated which will lead to improved knowledge of the seismicity of the Eastern United States. This knowledge may lead to possible future modifications to the seismic loading discussion presented in Chapter 3 of ATC-14. The following chapter of this report will also present a discussion of the seismicity issues raised during this project which may lead to future ground motion and mapping studies by NCEER researchers.

The present ground motion studies being performed are in the developmental stages and therefore probably will not be completed until at least sometime in 1989.

2.2 Suggested Topics for Possible Future NCEER Projects Which Could Improve ATC-14

During this review of the ATC-14 document, the project members identified a number of significant areas where they felt there was presently not enough information to properly address many of the issues required by the seismic evaluation procedure. A number of these issues are presently being investigated by other NCEER sponsored research projects. These projects, and the expected areas of improvement to ATC-14 which should result from the new information they will generate, were previously discussed in this Chapter.

But, a number of other issues identified during this review as areas of needed research are presently not being investigated by NCEER or other research institutions. These issues will be listed in this section with the intention that they will be considered as possible topics for future research projects.

The list of suggested areas for future research is broken up into seven sections, one for each of the six construction material types identified in ATC-14, and one for non-structural elements:

1. Wood Construction -
   a. A comparison between Eastern and Western United States home dwelling construction practices may point out differences which could affect the seismic resistance of these structures.
   b. The capacity and ductility of non-plywood wall framing elements.
   c. Seismic capacity and behavior of nailed connections, particularly at diaphragms.

2. Steel Construction -
   a. The capacity, stiffness and ductility of bolted tee and other typical moment connection details are not well understood.
   b. Riveted, bolted and welded splice connection details, especially to jumbo column sections.
   c. The ductility of "ordinary" moment frames. How does the response differ from that of a "ductile" moment frame?
   d. The capacity and ductility of the anchorage of infilled masonry to steel frames for both in-plane and out-of-plane forces.

3. Concrete Construction -
   a. Strength and ductility of reinforced concrete elements and connections that have not been designed for significant lateral forces.
   b. Construction joint shear capacity at wall-slab interfaces.
   c. Capacity and ductility of the anchorage of infilled masonry to concrete frames for both in-plane and out-of-plane forces.
   d. Ductility of "typical" connections at the intersection of walls and roofs or floor slabs.

4. Precast Construction -
   a. Capacity and ductility of connections between topping slabs, precast planks and concrete or masonry wall elements.
   b. Capacity of non-ductile wall panel and diaphragm details.
   c. Diaphragm behavior of untopped precast concrete floor and roof systems.
   d. Ductility of "typical" connections at the intersections of walls and roofs or floor slabs.

5. Reinforced Masonry Construction -
   a. Performance of connections, anchorages, and details in masonry structures, including connections of infill walls to framing members.

6. Unreinforced Masonry Construction -
   a. Capacity of stone masonry and rubble walls.
   b. Capacity and ductility of anchorages details for parapets, appendages, etc.

7. Non-Structural Elements -
   a. Stability of poorly supported interior masonry partitions.
   b. Capacity of elements such as partitions, ceilings, etc., typically used in buildings.
   c. Reliable connections for attaching facades, trim, veneer, and curtain walls to masonry and reinforced concrete structures.
   d. Strength and ductility of brick veneer/steel stud cladding wall system for out-of-plane loads.
   e. Strength and ductility of stone cladding and common attachments for out-of-plane loads.

f. General Cladding Issues:

- effect of insufficient isolation joints between cladding elements or between cladding and steel frame
- develop criteria for level of shaking for which isolation should be provided; this may vary for frame and cladding types
- evaluate performance of non-ductile anchors commonly used cladding anchors: powder driven fasteners, plastic window/curtainwall components, masonry embedments
- develop a more rational set of Cp factors for evaluating out-of-plane performance of wall cladding

CHAPTER 3
SEISMICITY ISSUES

Chapter 3 of the ATC-14 document, titled "Seismic Loading", presents the basic information necessary to develop the response spectra which the original project engineering panel deemed to be appropriate for the evaluation of existing buildings. Procedures were also developed for modifying the spectra to reflect different probability levels (return periods). A short discussion of the expected duration of strong ground shaking is also presented. This procedure incorporated the seismic zoning maps developed for the ATC-3 project.

During their review of this Chapter of the ATC-14 document, the review panel identified two major subjects where they felt significant improvements could be realized. These two subjects are the applicability of the present seismic zoning maps and the possibility of structural damage caused by effects other than ground shaking.

3.1 Applicability of the Seismic Zoning Maps

During the initial meeting of this project, the review panel discussed the applicability of the present seismic zoning maps for A_A and A_V in the Eastern United States. It was felt that there is a recently developed large body of knowledge concerning Eastern seismicity which has yet to be properly incorporated into these maps. As a result, this developing knowledge is not being utilized by practicing structural engineers.

In addition, the review panel noted that these seismic zoning maps are of critical importance to the emerging seismic safety programs of the Eastern United States. These maps form a principal criteria document for both earthquake engineering design and seismic safety policy. Both public awareness and professional information demands are rapidly increasing. These factors identify a compelling need to provide and maintain a definition of national seismic hazard zoning which continuously incorporates all of the rapidly developing knowledge in this area.

As a result of these discussions, the members of the project team concluded that there was an immediate need to begin the work on better defining the seismicity of the Eastern United States. They proposed to convene a meeting which would be held in conjunction with the New York Academy of Sciences Conference on Earthquake Hazards and the Design of Constructed Facilities in the Eastern United States held in New York City during February of 1988. This Conference, which was co-sponsored by NCEER, convened a large group of both scientists and engineers in an effort to assess the seismic hazard in the Eastern United States and the alternative policies for the engineering design community and related regulatory agencies in response to these hazards.

The meeting was attended by fifteen engineers and seismologists. The following five topics were discussed at this working group meeting:

1. Specific areas where it may be possible to update the present seismic zoning maps.
2. The most recent information on recurrence intervals for the Eastern United States.
3. The effects of distant earthquakes and duration on the seismic hazard in the Eastern United States.
4. Gaps in the present state of scientific knowledge regarding these issues.
5. Suggestions for specific research tasks which could be useful in bridging these gaps in our knowledge.

The minutes for this meeting, including the list of attenders, is included in Appendix B. Listed below are the major review comments which were generated during this meeting:

1. The seismic zoning maps presented in the document are those developed by Algermissen and Perkins in 1977 and updated in 1982. Through the results of the EPRI work on source modeling, a great deal more information is presently available on Eastern seismicity. The EPRI model could be used to develop an entirely new seismic zoning map for the Eastern United States. However, these maps should not be altered in local regions because of the need to reconsider the entire Eastern region. Local modifications to the maps would be difficult to perform except in the context of a regional study.

2. More recent information could cause significant modifications to some areas of the present ATC-14 maps. These areas include the following:

   a. Maine, near the Canadian Border
   b. Ohio
   c. Parts of South Carolina

3. A more explicitly probability-based procedure which includes the uncertainties in all the parameters could result in a more rational basis for determining the seismic loading. This would provide the engineer with more information to be used in reaching decisions.

4. The 475-year return period as the basis of the evaluation should be retained in order to be consistent with other design criteria. This return period may not be the most appropriate for other areas of the country.

5. There is a body of recently developed information on the effects of distance and duration of Eastern United States earthquakes which could be incorporated into Chapter 3 of ATC-14. Lamont-Doherty will begin work on these issues.

6. As presented, Chapter 3 of ATC-14 does not present all of the background information which was used to develop the recommended procedures.

As a result of this meeting, the participants formulated the following recommendations for continued work in this field which would assist in incorporating the latest possible information into the seismic zoning maps:

1. NCEER should fund a study to develop a set of seismic zoning maps for the Eastern United States using the EPRI source model. The results of this work should be coordinated with the present USGS project which is updating the existing maps. A decision concerning which (and how many) parameters should be mapped should be done through a coordinated effort of engineers and seismologists.

2. The EPRI model should also be used to study the effects of differing recurrence intervals on the seismic zoning maps. If the form of the maps do not change for different recurrence intervals, an approach similar to that presented in Figure 3.8 of ATC-14 may be appropriate.

3. The effects of distance and duration should be incorporated into the studies recommended above. Lamont-Doherty should coordinate work on these issues.

4. Complete documentation of the procedures used to develop the information presented in Chapter 3 of ATC-14 should be published. This documentation could be in the form of a technical paper.

3.2 Secondary Seismic Hazards

The other topic which the review panel felt would be an area of concern for structural engineers performing seismic evaluations of existing buildings is the potential for damage caused by secondary seismic hazards. They suggested that a discussion of these secondary hazards be added to Chapter 3 of ATC-14. The discussion in the following paragraphs presents some background information for the various secondary hazards and a more detailed description of the liquefaction phenomenon. Appendix B presents a preliminary procedure for the evaluation of liquefaction potential.

3.2.1 Introduction

Ground shaking is the principal seismic hazard considered in the design of earthquake resistant buildings. In somewhat less general circumstances, the collateral or secondary damaging effects of earthquakes can also present serious risk to human lives. These collateral effects are ground rupture in fault zones, ground failure, tsunamis, and fire. Of these, ground failure relates to the objectives and methodology of ATC-14. Like ground shaking, its effects are felt during the earthquake and are reasonably mitigated by structural measures common to specialized earthquake resistant design. The potential for damage by inundation, fire and direct contact with faulting can be assessed using the methodology provided in ATC-13, "Earthquake Damage Evaluation Data for California", (ATC, 1985).

Ground failure induced by earthquakes can occur as settlement, landslides and liquefaction. Loose dry sand and backfills can densify when vibrated during a seismic event, resulting in significant surface settlement. Sensitive clays or normally consolidated fine-grained soils can suffer a dramatic strength loss when strained by earthquake shaking. Shear deformation leads to initial strength loss which quickly transitions to catastrophic failure as plastic deformation is initiated. Lateral spreading and flow failures have occurred in sensitive clays. Whereas loose dry sands and sensitive clays have been known to cause significant damage, loose uniform sands saturated with water can cause far greater damage by losing their foundation support capacity through a process known as liquefaction.

This section will provide a summary discussion of the basic process of liquefaction, the types of liquefaction failures, some characteristic building structural damage, the relevance of liquefaction to ATC-14 and a practical means of insitu testing to screen local soils for liquefaction hazard. If these preliminary results are positive, the methodology recommends that the evaluating structural engineer then arrange for the assistance of a qualified geotechnical engineer in completing the evaluation of the existing building and its foundation.

3.2.2 Basic Process of Liquefaction

As discussed by Clough (1988), liquefaction is a phenomenon where a saturated soil, usually a sand, is subjected to a loading that causes the pore pressures in the soil to increase, and the effective stresses to decrease to the point that the soil can undergo large deformations under the actions of the ground loading. In other words, a saturated cohesionless soil is abruptly transformed from a solid to a liquid state as a result of increased pore pressure and loss of shear strength. When liquefaction is combined with conditions such as ground slope, surface loads, and the ejection of water and sediments, permanent movements develop such that structures supported or surrounded by soil can be severely damaged. Three conditions are necessary for liquefaction to occur. First, the earthquake must be at least a Richter Magnitude 5.5 event to generate noticeable liquefaction. Second, the groundwater table must be high. A groundwater table below 30 or 40 feet will likely suppress liquefaction. Third, the soils must be liquefiable. The most susceptible are fine sands with gravels, course sands and silty soils being more resistant to liquefaction, but still potentially liquefiable. A plot of grain size data for soils indicating those most likely to undergo liquefaction is shown in Figure 3.1 (Ishihara, 1985).

3.2.3 Types of Liquefaction

In considering the structural vulnerability of a building system to liquefaction, it is useful to consider the various forms in which it can occur. According to Youd (1983), these include the following six distinct types of ground failure:

Flow Failures are the most catastrophic ground failure caused by liquefaction. Flows can move relatively long distances, tens of feet to miles, at relatively high speeds that may reach tens of miles per hour. They may involve completely liquefied soil or blocks of intact earth riding on a layer of liquefied soil. They usually develop in loose, saturated sands and silty sand on slopes of 5 percent or more.

Lateral Spreads are the most common ground failure generated by liquefaction. They involve primarily lateral movement of surficial soil layers over a liquefied layer. These failures generally develop on very gentle slopes (most commonly between 0.5 percent and 5 percent). They involve lateral displacements ranging up to several feet, and in particularly susceptible conditions several tens of feet, accompanied by ground cracks and differential vertical displacements.

Slumps commonly occur in steep banks, particularly river banks, underlain by liquefied sediment. Vertical displacements are typically a large fraction of the height of the bank, and the width of the failure may be several times the height of the bank.

Loss of Bearing Strength allows heavy structures to differentially settle, settle or tip, and lightweight, buried structures to rise buoyantly.

Transient horizontal oscillation of the ground surface accompanied by ground fissures and differential settlement occur as a consequence of liquefaction of a layer at shallow depth beneath a level surface. The weakened layer decouples the surface layer from the underlaying firm ground, allowing the surface layer to oscillate in a different mode during continued earthquake shaking.

Sand Blows, although not strictly a form of ground failure, may cause damage through flooding and sedimentation. Sand Blows develop as a consequence of high porewater pressures generated during the liquefaction process. Dissipation of these pressures commonly occurs in transient eruptions that spurt water laden with sediment to the ground surface, causing local flooding and leaving the area spotted with irregular deposits of sand and silt.

3.2.4 Some Characteristic Building Structural Damage

The structural consequences due to liquefaction range from very severe or catastrophic to negligible depending on the degree and extent of liquefaction-induced ground failure. It is useful to consider a few examples from this wide range of building damage. The more severe type of structural damage includes settlement and severe tilting and overturning of structures. Figure 3.2 provides a well known photograph of severe tilting and settlement that resulted during the Niigata earthquake of 1964. The Great Alaskan earthquake of 1964 caused massive liquefaction induced flow failures that carried away major portions of three towns. The Puget Sound earthquake of 1965 had additional examples of liquefaction-caused damage, most notably the failure of quay walls caused by liquefaction of soils behind the walls. Liquefaction can also degrade piling performance by reducing skin friction. Furthermore, pile buckling resistance may become acute because of lack of lateral support in liquefied soils and downdrag loads caused by settlement after liquefaction.

General settlement can also lead to serious differential settlement of buildings. Figures 3.3 and 3.4 are of damage caused during the Nihon-Kai-Chubu, Japan Earthquake of 1983 and provide additional vivid illustration that liquefaction can be the primary destructive effect rather than ground shaking. Figure 3.3 shows the settlement of soil about 16 inches as seen from a school building which is on piles. Figure 3.4 shows the inside of a pile-supported terminal building which had its slabs on grade settle over 4 feet, as shown by one of the investigators standing in the resulting pit.

Failure can also be more localized, causing differential settlement even when the building is not well supported by piles. A simple wood frame house standing at a large sink that suffered such damage during the Charleston, South Carolina earthquake of 1886. As reported by McGee (Peters 1986):

> "Within the sink there have been swallowed up two of the brick piers supporting the house shown in the picture, and a peach tree 6 or 8 feet high with the exception of its topmost twigs. At the time of photographing, the water was sounded to the depth of 4 or 5 feet without finding bottom; and it was reported by the proprietor of the house and the adjacent store, Mr. Lee, that during the morning following the earthquake attempts were made to find the piers with a 15 foot pole, but that the bottom was not reached. It will be observed...that the building itself as well as the piers and chimney, were but slightly affected by the earthquake-indeed the chimney is not at all affected save that it has been shifted an inch to the westward (toward the sink) as indicated by a crevice at the eastern side, and slightly fissured at one point. This sink occurs on the margin of an extensive craterlet area from which great quantities of sand have been extravasated - the sand indeed extending over an area of 2 or 3 acres".

Proper seismic design depends upon a proper modeling of how a structure will move during an earthquake and adequate provisions for passage of lateral forces over a continuous path from the roof down to the foundation. Ground shaking is assumed that will cause lateral motion of the building in a back and forth direction. Vertical accelerations and motion exist but are rarely considered since only vertical load-carrying members may be affected. Dependence is placed on the large vertical load carrying reserves buildings generally have due to code gravity load requirements. As a consequence, typical buildings are notably vulnerable to the relative displacements, settlement and tilting of its foundation potentially caused by liquefaction. When significant liquefaction potential exists, specialized geotechnical engineering is necessary to properly assess the vulnerability of the existing building and/or to mitigate the liquefaction potential.

Considering the range in types of possible structural damage suggested above, it is apparent that structural design measures for liquefaction mitigation are additive to those provided for earthquake ground shaking hazard reduction. As discussed by Lew (1984), this is done by designing the building to withstand the added forces and deformations that would be likely to occur in the event of liquefaction. If some type of floating foundation was not preferable, piles could be used to transfer the building loads down into deeper non-liquefiable soils. The piles would be designed to withstand possible buckling due to the reduction in lateral support in liquefied soil and downdrag forces resulting from settlement. Floors on grade may require structural support and enhanced grade beams and tie beams may be necessary to preclude excessive differential settlement. Connections between structural members would also require special strengthened design. These recommended measures for new construction suggest that similar upgrades would be necessary in the case of existing buildings.

3.2.5 Relevance of Liquefaction to the ATC-14 Methodology

Liquefaction potential should be included in the ATC-14 methodology for several major reasons. First, significant hazard exists in many regions of the United States. The San Francisco earthquake of 1906, the great Alaskan earthquake of 1964, and the Puget Sound earthquake the following year provide many examples of liquefaction damage. In addition, extensive amounts of liquefaction have occurred during earlier events elsewhere, particularly in the New Madrid, Missouri earthquake of 1811-12 (Fuller, 1812 and Nuttli, 1981) and the Great Charleston, South Carolina, earthquake of 1886 (Dutton, 1889). Recent studies have revealed significant sand-blow structures along the South Carolina coastline from Beaufort to Myrtle Beach as shown in Figure 3.6, some dating before 1886 (Gohn, 1984). The multiple pre-1886 Holocene earthquake-induced liquefaction events have occurred within the last 7,200 years and document that at least three prehistoric liquefaction-producing earthquakes (M_b approximately 5.5 or larger) occurred during this period before the great earthquake of 1886.

The second reason for concern is that the structural damage caused by liquefaction can be severe, even more so than that caused by ground shaking. Mr. Harry O. Wood in contributing to the Report of the California State Earthquake Investigation reported that, in the San Francisco earthquake of 1906, "apparently five or ten times greater proportional damage to structures built on the soft, moist sands and sediments near the shoreline or on filled-ground over old swamps, than in similar buildings less than one-half mile away, built on hard ground or thinly covered projecting ridges of rock" (Freeman, 1932). In reporting on the Nihon-Kai-Chubu, Japan earthquake of 1983 (magnitude 7.7), Bertero (1985) noted that most of the damage observed to all types of structures was caused by ground failure as a result of the liquefaction of the subsurface soils on which the facilities were supported. Liquefaction caused 900 houses to collapse, with 2000 being severely damaged. There were about 750 other buildings damaged. In comparison, structural damage caused by ground shaking was comparatively light. Both the Niigata earthquake and the Great Alaska earthquake of 1964 featured spectacular liquefaction-caused damage.

The third reason for concern stems from past disregard. As observed by Stratta (1987), despite its major damage potential, liquefaction has not been a major concern in design in the United States. Relatively few existing buildings in the United States have been constructed to resist earthquakes. And even fewer of those in liquefaction-prone locations have been specifically designed to resist such collateral effects.

3.2.6 Screening Procedure for Liquefaction Potential

Recently developed information allows the geotechnical engineer to determine the liquefaction potential at a given site. In the case of new construction, the information can be used to design a structure and its foundation that is suitably resistant to liquefaction effects or to stabilize the soil itself. Because of uncertainties and the high costs of such measures, it is sometimes best to avoid sites with a potential for liquefaction when permitted by other relevant factors.

Because of the wide range in possible foundation options in existing buildings, the ATC-14 methodology should recommend that the liquefaction potential at a site be assessed and, if found to be positive, that the technical problem be referred to a qualified geotechnical engineer for resolution. Liquefaction potential maps can be used for that purpose as available for Charleston, South Carolina (Elton and Hadj-Hamou, 1988), Memphis, Tennessee (Nowak and Berg, 1981) and Massachusetts (Soydemir and LeCount, 1981). Commonly, such maps are not available. In these cases, when suspect, the liquefaction potential of the specific site must be evaluated as part of the existing building investigation.

The basic procedure for evaluating liquefaction potential is presented in Appendix B. The procedure is as described by Clough (1988) and Elton (1988) and is based on developments of others (Seed and Idriss, 1982; Seed and De Alba, 1986; Marcuson and Bieganousky, 1977).

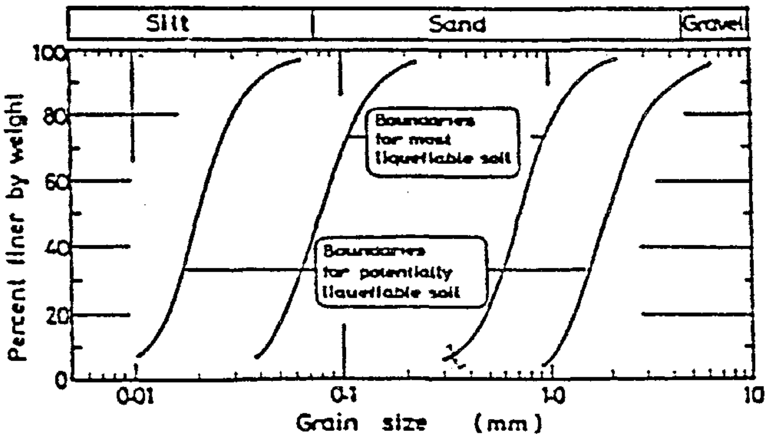


Figure 3.1 - Soil Potential for Liquefaction
(Ishihara, 1985)

Figure 3.2 - Building Damaged by Soil Liquefaction
(Seed and Idriss, 1982)

Figure 3.3 - Soil Settlement Due to Liquefaction
(Bertero, et. al.,1985)

Figure 3.4 - Slab on Grade Settlement Due to Liquefaction
(Bertero, et. al., 1985)

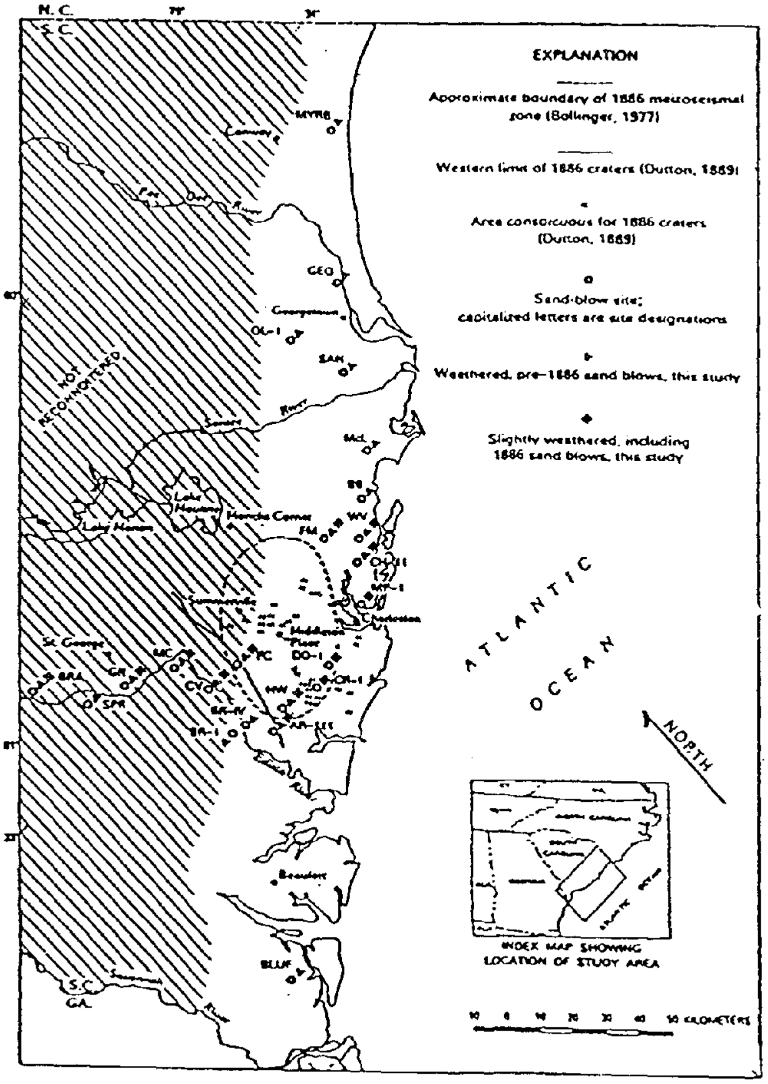


Figure 3.5 - Map of 1886 and Pre-1886 Sand
Blow Sites in South Carolina
(Gohn, et. al., 1984)

CHAPTER 4

REGIONAL DIFFERENCES

Another feature of the seismic evaluation of existing buildings which the review panel felt should be developed as a portion of this project were the regional differences between the Western United States and the rest of the country. These differences, which include such items as seismicity, building code requirements, and construction practices all can have a profound effect on the seismic resistance of an existing building. The following paragraphs provide a brief description of some of these differences. This description should provide the Engineer with additional insight which will be useful in performing the seismic evaluation.

4.1 Introduction

Design and construction practices, earthquake ground motions and a number of other differences exist between the eastern and western United States. These differences were considered in the development of the ATC-14 methodology in an effort to ensure that they reasonably apply to existing building evaluation in all seismic zones of the United States. These differences were maintained while restricting the regional categories to those of high seismicity and low seismicity. The principal regional differences include those of (1) seismicity including attenuation, (2) expectancy maps and risk analysis, (3) public awareness, (4) building code application, (5) training and experience of design engineers and code enforcement officials, (6) companion wind threat, and (7) age and weather environment of buildings. This section summarizes these differences to provide additional background information which could be useful in performing the seismic evaluation.

4.2 Differences in Seismicity

There are both similarities and differences between western and eastern United States earthquakes. Similarities exist in their depth of focus which normally ranges from 5 to 15 km and in their faulting which seems to be characteristically strike-slip in nature. While the latter is most common, both areas have exhibited reverse and normal faulting. As described by Nuttli (1982), the differences include their frequency of recurrence, source characteristics, magnitude-rupture length relations and attenuation. Complicating the proper consideration of these factors is the minimal instrumental data available for eastern earthquakes. This requires the adaptation of data acquired in other areas, principally the western states, a process subject to serious differences in professional judgment and requiring particular care, especially due to the differences in the earthquake source characteristics and in the efficiency of transmission of wave energy in the frequency range of damaging ground motion.

Large earthquakes in the eastern United states have occurred less frequently in this century than in the nineteenth century. However, major earthquakes have occurred during the eighteenth century and before. Further, continuing minor-to-moderate seismic activity occurring in several earthquake source zones is consistent with the occurrence of future major earthquakes in those zones. Figure 4.1 shows the epicenters of the largest earthquakes (i.e. M_b equal to or greater than 4.5) in the region east of and including the Appalachians. Although the epicenters of the largest earthquakes affecting the northeast region are located within the St. Lawrence Valley of Canada, including the November 1988 Chicoutimi Earthquake, and the number of moderate-sized earthquakes occurring within it has been small, the possibility of a major earthquake within the northeast region cannot be ruled out due to its seismicity being unknown for the period preceding the twentieth century. Considering historical seismicity including the 1755 Cape

An earthquake (epicentral intensity of between MM VII and MM VIII), an earthquake of $M_s = 6.5$ and a peak ground surface acceleration of 0.12g has been established as the design earthquake for Massachusetts and considered representative for New England states by important regional technical groups (Soydemir, 1986). Seismicity in the southeastern United States contains an active zone near Charleston, South Carolina but appears to be primarily associated with the Appalachians. The recurrence time of an 1886-size earthquake was reported by Nuttli as being 1,000 years, but an improved estimate should result from continuing studies of discovered pre-historic sand-blow sites along the coast of South Carolina. Figure 4.2 shows the moderate-level historical seismicity of the central United States. All of the earthquakes can be associated with major geological structures, of which only the New Madrid fault zone has the potential to produce earthquakes of M_s of 8.0 or more. The earthquake activity is principally located within the lower Ohio river valley and the central Mississippi regions. During the winter of 1811-1812, three great earthquakes occurred in the New Madrid fault zone, all of surface-wave magnitude M_s above 8.

Whereas they occur more frequently, the damaging ground motion of western earthquakes is attenuated at rates much greater than their eastern counterparts. The attenuated ground motion is that of the most damaging earthquake waves in the frequency range of 0.1 to 10 Hz which travel in the upper layer of the earth's crust. The latter extends to a depth of about 20 km. Figure 4.3 illustrates a comparison of the structural and architectural damage zones for several large eastern and western earthquakes. Note that the New Madrid earthquakes of 1811-1812 had areas of structural damage (i.e. MM VIII or greater) five times larger than that of the 1906 San Francisco earthquake, and areas of nonstructural or architectural damage (i.e. MM VI and VII) twenty times larger. All of these earthquakes had surface-wave magnitudes exceeding eight, which classifies them as great earthquakes.

Associated with the attenuation differences is an important change in duration of shaking. As distances increase beyond the near-field region, the dispersion of the surface waves becomes dominant, causing the ground-motion duration to increase with distance. This is particularly evident for frequencies of 1 Hz and less.

Lower period waves have natural frequencies close to that of low-rise buildings, affecting such structures nearer the epicenter. Whereas, long period waves travel farther and can affect the taller buildings which oscillate at comparable frequencies. These near-sinusoidal, long period motions will be particularly noticeable in the upper levels of high-rise buildings, because the frequency of the ground motion will be close the resonant frequency of the buildings. Non-structural elements and contents are particularly vulnerable to such earthquake shaking.

The damage potential of these long period waves (as well as those of lower periods) can be further aggravated by site amplification as discussed by Hayes (1986) and demonstrated by the 1985 Mexico earthquake. Site amplification depends upon several factors including the fundamental natural period of the soil-column. The increase or reduction of the building's response depends upon the relative magnitude of the building's natural period of vibration to that of its supporting soil-column. The most critical situation results when they are equal, creating a condition of resonance which could result in severe damage.

In general, unlike their western counterparts, the eastern earthquakes do not rupture the earth's surface. Also, large magnitude eastern earthquakes do not have long fault lengths. Both of these characteristics make it far more difficult to delineate the earthquake source zones in the east. They also mean that in the East, relatively modest appearing mid-plate faults, if they are presently active, might produce large magnitude earthquakes. Accordingly, in the eastern region, earthquake source assessments must pay particular attention to moderate-size geological structures as well as the major ones. As discussed below, these differences notably influence earthquake hazard maps.

4.3 Expectancy Maps and Risk Analysis

Until fairly recently, ground shaking hazard maps in the United States have been based upon estimates of the maximum ground shaking experienced during the recorded historical period without consideration of the frequency of occurrence of earthquakes. This is a deterministic approach as contrasted with the probabilistic approach developed by Cornell (1968) and applied by Algermissen and Perkins (1976) and the Applied Technology Council (1976) in the production of current building code seismic hazard maps for the United States. They give the horizontal bed rock accelerations which have a 90% probability of not being exceeded in 50 years, the latter taken to be the life expectancy of the ordinary structure. This approach may not be as valid in the east as it is in the west because of the differences in recurrence rates previously discussed. It underestimates the hazards in those eastern regions that have a long recurrence interval for major earthquakes and have not experienced large earthquakes in historic time. Some of these areas are classified as "low seismicity" regions even though they will experience large earthquakes in the future. In general, the seismic hazard maps prescribe much higher accelerations in the west than in the east, reflecting in part the more rapid recurrence rates in western earthquakes, the associated success in identifying all the major earthquake source zones and the maximum-magnitude earthquakes associated with them from this historic record. As Nuttli (1982) discussed, this situation is particularly serious considering the likelihood of a low-probability large earthquake occurring somewhere in the eastern United States during the next 50 or 100 years whose ground motions will exceed those shown in these probabilistic seismic hazard maps. McQuire and Stepp (1986) examined a major study conducted by the Electric Power Research Institute of the seismic hazard in the eastern United States using nine test sites and confirmed the high level in uncertainty of hazard estimates. They further concluded that "the uncertainty of hazard estimates to be significantly less for sites located within the continental platform as compared to those sites located eastward of the western Appalachian Mountain System boundary". This uncertainty reflects poorly defined tectonic features and broad, regional interpretations of seismic sources.

As a result of this project initiative (as described in Section 3.1 of this report), the National Center for Earthquake Engineering Research (NCEER) plans to develop a set of seismic zoning maps for the eastern United States using the new information on eastern seismicity produced by the EPRI work on source modeling. The results of this work is to be incorporated with an ongoing USGS project to update the existing maps.

4.4 Public Awareness

The differences in regional seismicity have accounted for corresponding differences in seismic hazard reduction and mitigation programs of the eastern United States with those of the far more progressive western United States. On the west coast, most people have been exposed to moderate-sized earthquakes. This factor of greater frequency has resulted in strong local and state activities in western states to increase seismic technical knowledge and to promote the achievement of public seismic policy. The results have been commendable. Although significant technology development requirements remain, at least minimum seismic technical knowledge exists to support public awareness and safety policy establishment.

In contrast, little progress has been achieved in seismic hazard reduction and mitigation throughout the entire eastern United States. Whereas some notable advancements are beginning to be made, the seismic safety posture is that of California several decades ago. Public policy developments regarding seismic safety have been insignificant. The public is largely aware that earthquakes could occur but they do not understand the high levels of injury, death and property damage that could result or the prudent measures that could be taken to mitigate those effects. Eastern communities are not prepared to cope with the social and economic impacts from either a recurrence of a major earthquake or from smaller damaging earthquakes.

Public awareness is prerequisite to public appreciation of risk and commitment to the large scale mitigation efforts needed in the eastern United States.

This major difference in public awareness and commitment manifests itself in the relative status of construction adequacy for earthquake resistance. In the national regions of "low seismicity", there is generally no voluntary commitment to upgraded construction being that which may be prescribed and required by building codes. The ATC-14 document tries to establish a minimum acceptable level of safety for all existing structures.

4.5 Building Code Applications

As recently assessed by Beavers (1987), "the design and construction of buildings, or for that matter any type of facility, in the eastern United States for earthquake loads has been virtually nonexistent. Only nuclear power plants have been consistently required to be designed for earthquake loads (seismic design), even though the threat is real for all facilities." This matter of considerable concern as 80% (194 million) of the U.S. population (242 million) live in the eastern and central United States with 60% (145 million) live east of the Mississippi River. Until recently, only three areas of the eastern United States are known to have adopted some recognized form of mandatory seismic design into the governing building codes for new buildings or facilities. They are the states of Massachusetts (1974) and Kentucky (1983) and the city of Charleston, South Carolina (1983). Note also that many branches of the Federal Government such as the Veterans Administration, the General Services Administration, the Navy, etc. also have mandatory seismic requirements for all their facilities. In 1988, the Standard Building Code (SBC) was amended to include seismic design requirements as part of its basic provisions. All municipalities in the southern and southeastern United States requiring the SBC will now be mandating seismic design. With this major advancement, all three major model building codes used in the United States now include some form of seismic design requirements. However, given these historical developments, it should be assumed that essentially all existing buildings in the eastern United States were constructed without the incorporation of seismic design measures. Particularly in the coastal regions, buildings code wind provisions have been applied in probably most cases to varying degrees of effectiveness. Section 5.3 identifies the building code adoption status of all states. As a consequence, the engineer should anticipate that any seismic strength of existing buildings in the eastern United States will primarily be the product of good conventional design and construction practices and, sometimes, applied wind design considerations, rather than a deliberate seismic design.

4.6 Training and Experience

As another result of relatively low regional seismicity and the absence of seismic building code requirements, technical information and trained capability regarding earthquake engineering are extremely limited throughout the eastern United States. Very few of the professional engineers practicing in the eastern United States have been educated or experienced in seismic design. Few, if any, professional engineering registration examinations include seismic design exercises. Seismic policy, public awareness, and prudent building code standards and practices are all impeded by this technology inadequacy. There is a similar need to educate building officials regarding effective enforcement of building code seismic design and construction practices. These needs are now widely recognized and are being addressed. However, it is important to recognize that in the eastern United States, the evaluation guidelines will be largely applied by qualified professionals who are familiar but not yet expert in earthquake engineering.

4.7 Companion Wind Threat

Certain eastern buildings exist within high wind hazard regions and were constructed, to varying degrees of design and construction adequacy, with certain levels of lateral force resistance. These buildings will therefore have some inherent seismic resistance, but may not have the necessary ductility required to resist the expected seismic overloads. The engineer should be especially careful to check the member connections to determine if they have been provided with sufficient ductility. Much of the evaluation procedure presented in ATC-14 is intended to address this issue.

4.8 Age and Weather Environment of Buildings

The engineer should anticipate certain structural deterioration of existing buildings due to widely ranging climate and other environmental conditions. There is particular concern regarding non-structural components such as cladding components. The greater age of many of existing buildings in the eastern United states should also be considered. Unreinforced masonry buildings provide an example. These were found in Coalinga, California of design similar to their historic counterparts in Charleston, South Carolina. However, the Charleston buildings were much older, many had been earthquake damaged, and some even endured great fires. A number of additional statements and procedures addressing this topic have been developed in this project for inclusion into ATC-14. These statements are included in Chapter 6 of this report.

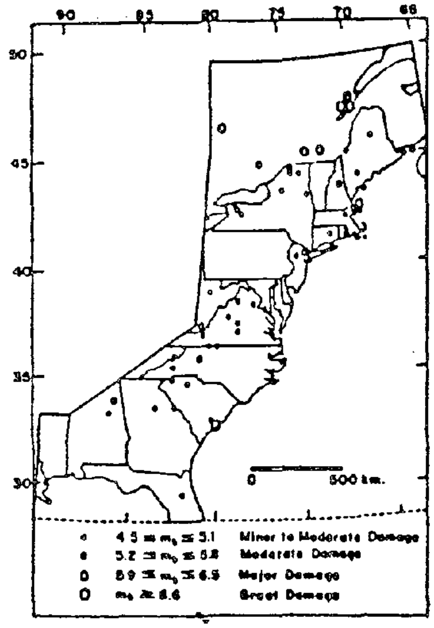


Figure 4.1 - Earthquakes of M_b Equal to or Greater than 4.5 for the Region East of and Including the Appalachians

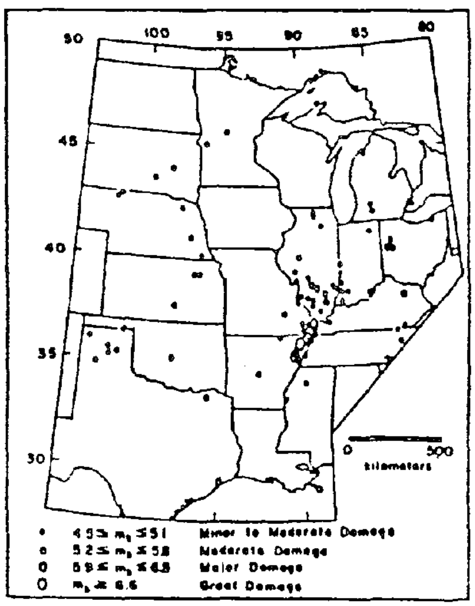


Figure 4.2 - Earthquakes of M_b Equal to or Greater than 4.5 Between the Rocky Mountains and the Appalachians

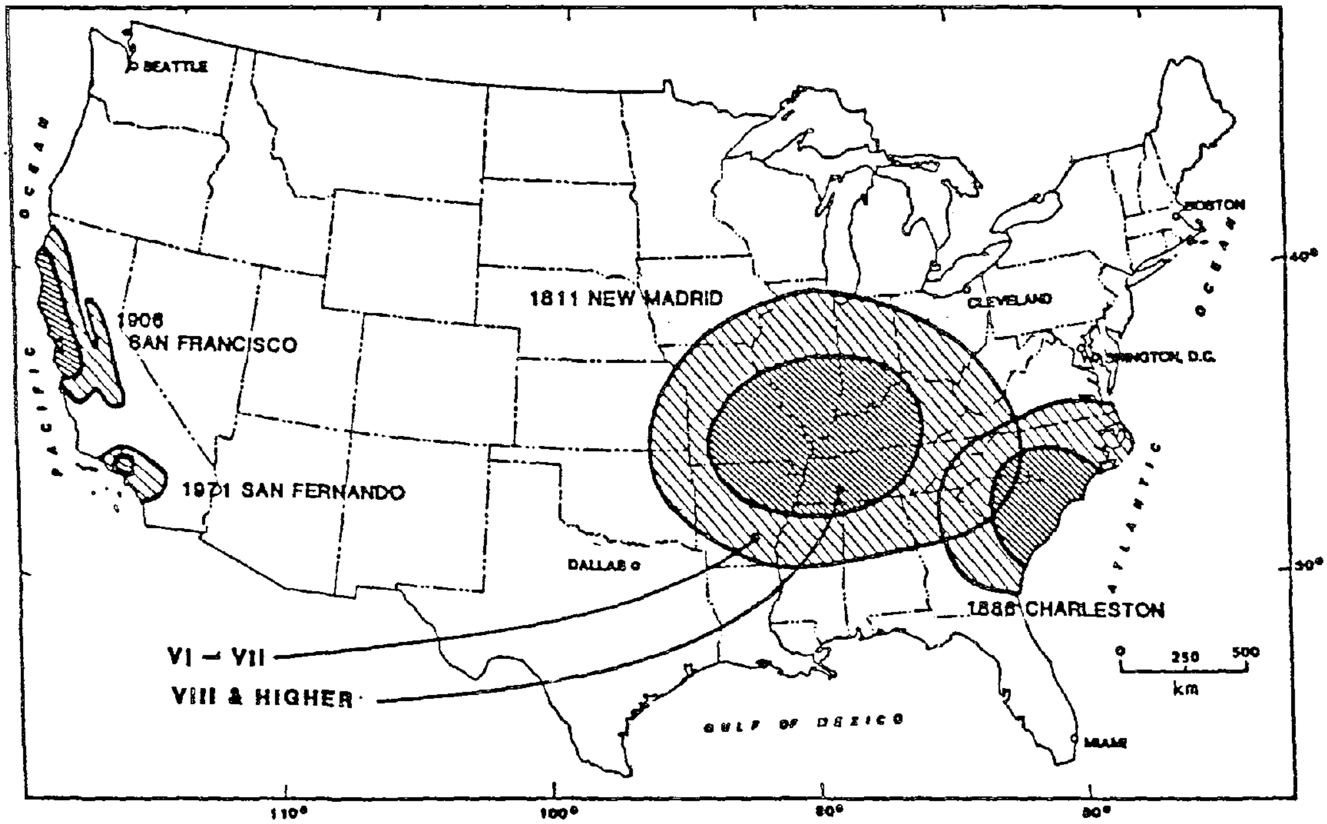


Figure 4.3 - Comparison of Isoseismal Contours for Earthquakes: San Francisco (1906), New Madrid (1811-1812), San Fernando (1971), and Charleston (1886)

CHAPTER 5

SUGGESTED ADDITIONS TO THE ATC-14 LISTS OF REFERENCES AND STANDARDS

ATC-14 includes an extensive set of references which were reviewed and/or used in the development of the document. These references were cited to provide the user with a means to access more information than could be incorporated in the evaluation procedure. In order to facilitate its use, these references were categorized and listed together by topic. The following topic headings are used in this section:

1. Earthquake Damage Reports
2. Existing and Proposed Code Provisions
3. Evaluation Methodologies and Examples
4. General Papers on Analysis and Retrofit Procedures
5. Wood Structures
6. Steel Structures
7. Concrete Structures
8. Precast or Prestressed Concrete Structures
9. Masonry Structures
10. Unreinforced Masonry Structures
11. Testing Methods

A total of over 250 references are listed in the document.

During their review, the panel members identified a few areas which they felt were not adequately addressed in the references to ATC-14. It was felt that collecting and cataloging a more complete list of references and/or information on these topics could provide further assistance in the seismic evaluation process. References and information which were collected to supplement those provided in the ATC-14 document address the following topics:

1. Historical references which contain information on the methods of design and construction typically in use before the introduction of modern codes.
2. Reference standards for all construction materials, including mailing addresses.
3. Information on the adoption of model building codes by states and local municipalities.
4. Examples of building performance from earthquakes in the Eastern United States.

The lists of this supplemental information are presented in this Chapter.

5.1 Historical References

A large number of buildings, especially in the Eastern United States, were constructed before the introduction of modern building codes. Also, many older buildings were designed and constructed by "Master Builders", often without the assistance of complete drawings. Often contractors may have developed proprietary methods of construction for which they obtained patents. Until recently, many municipalities did not obtain construction documents for their files. Over time, construction documents are often lost or misplaced. These factors combine to make it difficult, if not impossible, to obtain complete construction documents for many older buildings.

If drawings are not available, the Engineer must resort to other means by which to obtain the information necessary to perform the seismic evaluation. Often, more extensive field work can be performed to determine much of this information. But, in many instances, such as in reinforced concrete construction, field investigations cannot provide adequate information unless expensive destructive investigations are performed. This alternative is often not feasible for a seismic evaluation project.

Recognizing these possible difficulties in obtaining sufficient information for the seismic evaluation of an older building, the Review Panel for this Project noted that another source of information could be any of the early books or papers on construction practice. These "Historical Documents" often include a wealth of information on the design, detailing and construction practices typically in use at the time of publication. A number of these documents were collected and briefly reviewed by members of this Project. These references, including a brief description and the Library of Congress classification number, are listed below. Table 5.1 summarizes this information:

1. **General Specifications for Steel Roofs and Buildings** by Charles Evan Fowler, 3rd Edition, Revised, 1897. Defines snow and wind loads, allowable stresses, etc. for steel, iron, and timber buildings. There are also sections on details of construction and on workmanship. (TH 2391)

2. **Building Code of New York City** by Mark Ash and William Ash (of the New York Bar), 1899. This rather detailed volume (nearly 200 pages) contains the 1899 New York City Code along with annotations and discussions. (TH 425)

3. **Building Code of New York City** compiled by William D. Brush, Assistant Superintendent, Borough of Manhattan, May 1, 1922. The code, which contains the many changes that were initiated in 1915 and 1916, was expanded in length to about 350 pages. (TH 225)

4. **Building Construction and Superintendence, Part I - Mason's Work** by F.E. Kidder, Architect, Various Editions, 1900-1920. This volume of 422 pages is rich with descriptive material and illustrations on foundations, stonework, brickwork, terra cotta, fireproofing, iron and steel supports for masonry work, lathing and plastering, and concrete building construction. Details appear to be "national" in that examples are drawn from California to New York. (TH 145)

5. **Building Construction and Superintendence, Part II - Carpenter's Work** by F.E. Kidder, Architect, Various Editions, 1900-1920. This book contains 535 pages of detailed information on all phases of building with wood, including chapters on wood framing (ordinary construction); sheathing, windows and outside door frames; outside finishing; interior woodwork, rough work, floors and stairs; builders hardware; and heavy framing. (TH 145)

6. **Building Construction and Superintendence, Part III - Trussed Roofs and Roof Trusses** by F.E. Kidder, Architect, 1906. This fascinating volume of nearly 300 pages is perhaps the most valuable of the three volume work by Kidder. It covers types of wooden and steel trusses, layout of trussed roofs (including bracing), open timber roofs and church roofs, vaulted and domed ceilings, octagonal and domed roofs, coliseums, armories, train sheds, and exposition buildings. It is full of illustrations of a number of famous (and not so famous) structures, many of which were originally published in the Engineering Record. (TH 145)

7. **Structural Designers Handbook** by W.F. Scott, 1904. This slim book is mainly for design, but it would also be quite useful in evaluating old structures. It has chapters on floor framing with steel members, spandrel beams, grillage beams, end reactions, steel columns, cast-iron columns, loads, allowable stress, brick walls, properties of foreign I-beams (German, Belgian and English), cast-iron bases and lintels, and wooden beams and posts.

8. **Structural Members and Connections** edited by George A. Hool and W.S. Kinne, 1923, 611 pp. This book is one of a six-volume series by Hool and Kinne. It has a wealth of detail on analysis, design of steel and cast-iron members, splices and connections for steel members, design of wooden members, splices and connections for wooden members, and design of reinforced concrete members. It is richly illustrated. (TK 260)

9. **Handbook of Building Construction, Volume I,** edited by George A. Hool and Nathan C. Johnson, 2nd Edition, 1929, 811 pp. This handbook was compiled by a staff of fifty specialists. It covers analysis , design and construction, estimating and contracting, and mechanical and electrical equipment. It is a good balance of theory, design, and detail. (TH 145)

10. **Pocket Companion for Engineers, Architects and Builders,** 21st Edition, 1920. The first edition of this "classic" handbook for designers of iron and steel structures was published in 1872. In addition to information on all types of members, the book has material on floor systems, connections, roof construction, etc. (TA 684)

11. **Steel Mill Buildings** by Milo S. Ketchum, 1906, 464 pp. In addition to much information on analysis and design, this book has a good deal of coverage of construction details. (TH 4511)

12. **Timber Design and Construction** by Henry S. Jacoby and Roland P. Davis, 2nd Edition, 1930, 334 pp. This book begins with 133 pages on fastenings and joints used in timber framing. It also has chapters on wooden beams and columns, wooden roof trusses, examples of framing in practice, and timber grades and allowable stresses. (TA 666)

13. **Handbook of Brick Masonry Construction** by John A. Mulligan, 1942, 525 pp. This book is devoted to a consideration of the various materials used in bricklaying and in brick-masonry construction, with many illustrations on "how to do it". It features New York City practice (Mulligan was President of the Associated Brick Masonry Contractors of Greater New York, Inc.).

14. **Handbook of Reinforced Concrete Building Design** by Arthur R. Lord, 1st Edition, 1929, 261 pp. Elaborate design aids for all types of reinforced concrete members are presented, along with reprints of the ACI paper on "Design and Cost Data for the 1928 Joint Standard Building Code" and the report of Committee E-1 on "Reinforced Concrete Building Regulations and Specifications". (TH 1501)

15. **Concrete Work: A Book to Aid the Self-Development of Workers in Concrete and for Students in Engineering** by William K. Hatt and Walter C. Voss, Volume 1, 1921, 451 pp. This very practical, well-illustrated volume includes chapters on simple footings, column footings, concrete walls, reinforced concrete frames, erection of reinforced concrete, pre-cast stone, walls and partitions, concrete walks and paving, building finish, special concrete constructions, and estimating. (TA 681)

16. **Concrete Building Construction** by Theodore Crane and Thomas Nolan, 1927, 689 pp. This book has considerable detail on concrete construction in addition to more general design principles and practices. (TA 683)

17. **A Treatise on Concrete: Plain and Reinforced** by Frederick W. Taylor and Sanford E. Thompson, John Wiley & Sons, New York, 1906. This book includes long discussions on concrete properties, placement, and construction. Details for beam-column joints, piles, and retaining walls are included. (TA 439)

18. **Historical Record, Dimensions and Properties: Rolled Shapes, Steel and Wrought Iron Beams and Columns as Rolled in U.S.A., Period 1872 to 1952, With Sources As Noted** by Herbert W. Ferris, American Institute of Steel Construction, Chicago, Illinois, 1983. This book contains the dimensions and basic properties of all steel and wrought iron sections produced in the United States between 1873 and 1952. Information on allowable yield and tensile stresses of the various steels provided during this period are also provided. (TA 685)

19. **Reinforced Concrete Construction** by George A. Hool, Volumes I - III, McGraw-Hill, New York, 1927. This book is a complete design text for concrete structures, including flat slab floors, continuous beams, columns, footings, etc. Few sketches are included. (TA 683)

20. **Design of Modern Steel Structures** by Linton E. Grinter, The MacMillan Company, New York, 1941. This book contains a lengthy discussion of riveted joints, including allowable stresses. Design of steel trusses and plate girders are also discussed. A chapter on wood design and detailing is also included. (TA 684)

21. **"Sweet's" Indexed Catalogue of Building Construction,** Beginning in 1906, Architectural Record Company, New York, 1906. This book contains information provided by builders on potential forms of construction. A number of proprietary systems for fireproof construction of steel framed buildings are presented. Other sections provide information on brick, terra cotta and other construction materials. (TH 12)

22. **Modern Connectors for Timber Construction,** National Committee on Wood Utilization, U.S. Department of Commerce, Government Printing Office, Washington, D.C., 1933. This document presents a wealth of information on timber connection techniques typically used for timber trusses such as split rings. Some basic allowable stress information for bolts is also provided. (TA 666)

TABLE 5.1 - LIST OF HISTORICAL DOCUMENTS

Author and Title of Reference	Date(s) of Publication	Wood	Steel	Concrete	Masonry
1. Fowler, General Specifications for Steel Roofs and Buildings	1897	X	X		
2. Ash and Ash, Building Code of New York City	1899	X	X	X	X
3. Brush, Building Code of New York City	1922	X	X	X	X
4. Kidder, Building Construction and Superintendence, Part I - Mason's Work, Various Eds.	1900-1920				X
5. Kidder, Building Construction and Superintendence, Part II - Carpenter's Work, Various Eds.	1900-1920	X			
6. Kidder, Building Construction and Superintendence, Part III - Trussed Roofs and Roof Trusses	1906	X	X		
7. Scott, Structural Designer's Handbook	1904	X	X	X	X

Table 5.1 (Continued)

Author and Title of Reference	Date(s) of Publication	Wood	Steel	Concrete	Masonry
8. Hool and Kinne, Structural Members and Connections	1923	X	X	X	X
9. Hool and Johnson, Handbook of Building Construction, Volume 1, 2nd Edition	1929	X	X	X	X
10. Pocket Companion for Engineers, Architects and Builders, 21 Editions	1872-1920		X		
11. Ketchum, Steel Mill Buildings	1906		X		
12. Jacoby and Davis, Timber Design and Construction, 2nd Edition	1930	X			
13. Mulligan, Handbook of Brick Masonry Construction					X
14. Lord, Handbook of Reinforced Concrete Design	1928			X	

Table 5.1 (Continued)

Author and Title of Reference	**Date(s) of Publication**	**Wood**	**Steel**	**Concrete**	**Masonry**
15. Hatt and Voss, Concrete Work: A Book to Aid the Self-Development of Workers in Concrete and Students in Engineering	1921			X	
16. Crane, Concrete Building Construction	1927			X	
17. Taylor and Thompson, A Treatise on Concrete: Plain and Reinforced	1906			X	
18. Ferris, Historical Record Dimensions and Properties: Rolled Shapes, Steel and Wrought Iron Beams and Columns As Rolled in U.S.A., Period 1872 to 1952, With Sources As Noted	1983		X		
19. Hool, Reinforced Concrete Construction	1927			X	
20. Grinter, Design of Modern Steel Structures	1941		X		

Table 5.1 (Continued)

Author and Title of Reference	**Date(s) of Publication**	**Wood**	**Material(s) Addressed Steel**	**Concrete**	**Masonry**
21. "Sweet's" Indexed Catalogue of Building Construction	1906		X	X	X
22. Modern Connectors for Timber Construction	1933	X			
23. Baker, A Treatise on Masonry Construction	1897				X
24. Crane, Architectural Construction, The Choice of Structural Design	1947 & 1956	X	X	X	X
25. Voss and Henry, Architectural Construction Volume I, An Analysis of the Design and Construction of American Buildings	1925	X	X	X	X
26. Voss and Henry, Architectural Construction Volume II Books 1 and 2, An Analysis of the Structural Design	1926		X	X	
27. Dietz, Dwelling House Construction	1954				

Table 5.1 (Continued)

Author and Title of Reference	**Date(s) of Publication**	**Material(s) Addressed**			
		Wood	**Steel**	**Concrete**	**Masonry**
28. Dunham, Foundations of Structure	1950				
29. Jacoby and Davis, Foundations of Bridges and Buildings	1956				
30. Michaelson, Leno, Industrial Inspection Methods	1950				

5.2 Reference Standards

Section 4.4.1 of the ATC-14 document addresses the reference standards to be used in the calculation of member capacities for the basic structural materials. Typically, this consisted of referring to the appropriate chapter of the Uniform Building Code. While this set of references provided guidance on the calculation of member capacities for most situations, a number of other reference standards could be useful. A more complete listing of appropriate material standards were therefore catalogued by the project team. This list, which is presented below, contains all of the information necessary to obtain each of these documents.

General:

Uniform Building Code and Uniform Building Code Standards
International Conference of Building Officials
5360 South Workman Mill Road
Whittier, California 90601
May, 1988

Building Standards, Evaluation Reports - Materials, Products, Methods and Types of Construction
International Conference of Building Officials
5360 South Workman Mill Road
Whittier, California 90601
1988

Standard Building Code
Southern Building Code Congress International Inc.
900 Montclair Road
Birmingham, Alabama 35213-1206
(205) 591-1853
1988

The BOCA National Building Code/1987
Building Officials and Code Administrators International Inc.
4051 W. Flossmoor Road
Country Club Hills, Illinois 60477-5795
(312) 799-2300

Wood: National Design Specification for Wood Construction
1250 Connecticut Avenue, N.W.
Washington, D.C. 20036
(202) 797-5900
January, 1986

Timber Construction Standard
American Institute of Timber Construction
333 West Hampten Avenue
Englewood, Colorado 80110
(802) 525-1625

Steel: Manual of Steel Construction
American Institute of Steel Construction
400 North Michigan Avenue
Chicago, Illinois 60611
(312) 670-5407
1980 - Allowable Stress Design, 8th Edition
1986 - Load and Resistance Factor Design, 1st Edition

Cold Formed Steel Design Manual
American Iron and Steel Institute
1000 - 16th Street, N.W.
Washington, D.C. 20036
(202) 452-7184
1986

Structural Welding Code - AWS D1.1-88
American Welding Society
P.O. Box 351040
Miami, Florida 33135
1988

Standard Specifications, Load Tables and Weight Tables
for Steel Joists and Joist Girders
1205 - 48th Avenue, North, Suite A
Myrtle Beach, South Carolina 29577

Diaphragm Design Manual
Steel Deck Institute
P.O. Box 9506
Canton, Ohio 44711
(216) 493-7886

Concrete: Building Code Requirements for Reinforced Concrete
ACI 318-83 and ACI 318M-83
American Concrete Institute
Box 19150 Redford Station
Detroit, Michigan 48219
(313) 532-2600
1983 with 1986 Supplement

Manual of Concrete Practice
American Concrete Institute
Box 19150 Redford Station
Detroit, Michigan 48219
(313) 532-2600

PCI Design Handbook - Precast and Prestressed Concrete
Prestressed Concrete Institute
201 North Wells Street
Chicago, Illinois 60606
1985

Post-Tensioning Manual
Post-Tensioning Institute
301 W. Osborn, Suite 3300
Phoenix, Arizona 85013
(602) 265-9158
1985

PCI Manual on Design of Connections for Precast
Prestressed Concrete
Prestressed Concrete Institute
201 North Wells Street
Chicago, Illinois 60606
1973

Masonry: Building Code Requirements for Engineered Brick Masonry,
Technical Notes
Brick Institute of American (Formerly Structural Clay
Products Institute)
11490 Commerce Park Drive
Reston, Virginia 22091
(703) 620-0010

Design Manual - The Application of Reinforced Concrete Masonry Load-Bearing Walls in Multi-Storied Structures
National Concrete Masonry Association
P.O. Box 781
Herndon, Virginia 22070
(703) 435-4900

Building Code Requirements for Masonry Structures
(ACI-ASCE 530)
American Concrete Institute
Box 19150 Redford Station
Detroit, Michigan 48219
(313) 532-2600
1988

Aluminum: Specification for Aluminum Structures
Aluminum Association
900 - 9th Street, N.W.
Washington, D.C. 20006
(202) 862-5100
December, 1986

5.3 Building Code Adoption Information

During the seismic evaluation of an existing building, useful information can be gained from the knowledge of the applicable building code, if any, which was used in the original design. This knowledge will provide the engineer with information such as the base shear and detailing requirements prescribed by the code for the design of the structure being evaluated. This information will provide insight that will facilitate the seismic evaluation.

Each state has jurisdiction over the adoption of a building code. A number of model building codes are presently prescribed by the different states. In addition, some states only require code compliance for state-owned or other special buildings, while in others, code compliance is completely mandatory. The date of adoption also varies from state to state. Table 5.2 lists building code adopted by each of the fifty states, the District of Columbia and the Virgin Islands. This table also includes the date of adoption and the status (mandatory, voluntary, etc.) of the code requirements. It is based upon information presented in the Fourth Edition of "Directory of State Building Codes and Regulations", prepared by the National Conference of States on Building Codes and Standards, in Herndon, Virginia.

It should be noted that major cities, such as New York, Atlanta, Los Angeles, San Francisco, etc., may have instituted their own building code which could vary from the state requirements. The evaluator should therefore also check the local building code requirements, in order to determine the code used in the original design of the building.

TABLE 5.2 - BUILDING CODE ADOPTION INFORMATION *

State	Date of Adoption	Mandatory/ Optional	Code
Alabama	1957	Mandatory	SBC
Alaska	1955	Mandatory	UBC
Arizona	---	---	None
Arkansas	1955	Mandatory for State Buildings Only	SBC
California	1951	Mandatory	UBC
Colorado	1971	Mandatory for Hotels and Motels	UBC
Connecticut	1971	Mandatory	Basic
Delaware	---	---	None
Florida	1975	Mandatory	SBC, 30.FLA, EPCOT
Georgia	1974	Voluntary	SBC
Hawaii	---	---	None
Idaho	1975	Voluntary	UBC
Illinois	---	---	None
Indiana	1973	Mandatory	UBC
Iowa	1970	Voluntary - Mandatory for State Buildings	UBC
Kansas	1968	Mandatory for State Buildings and Schools	UBC
Kentucky	1979	Mandatory	Basic
Louisiana	1986	Mandatory	NFPA 101
Maine	---	---	None
Maryland	1971	Voluntary - Mandatory for State Buildings	Basic

Table 5.2 (Continued)

State	Date of Adoption	Mandatory/ Optional	Code
Massachusetts	1972	Mandatory	Basic
Michigan	1974	Mandatory	Basic
Mississippi	1985	Mandatory for State Buildings Only	SBC
Missouri	---	---	None
Montana	1970	Mandatory	UBC
Nebraska	---	---	Legis. Pending for Basic
Nevada	1955	Mandatory for State Buildings Only	UBC
New Hampshire	1981	Mandatory for New Public Buildings	Basic
New Jersey	1977	Mandatory	Basic
New Mexico	1964	Mandatory	UBC
New York	1984	Mandatory Except NYC	NY State Uniform Fire Prev. & Bldg.
North Carolina	1935	Mandatory	NC State Bldg.
North Dakota	1982	Mandatory	UBC
Ohio	1979	Mandatory	Basic
Oklahoma	1981	Mandatory for State Buildings Only	UBC
Oregon	1974	Mandatory	UBC
Pennsylvania	---	---	None
Rhode Island	1977	Mandatory	Basic

Table 5.2 (Continued)

State	Date of Adoption	Mandatory/ Optional	Code
South Carolina	1972	Voluntary - Mandatory for State Buildings Only	SBC
South Dakota	---	---	None
Tennessee	1948	Mandatory	SBC
Texas	---	---	None
Utah	1985	Mandatory	UBC
Vermont	1981	Mandatory	Basic
Virginia	1973	Mandatory	Basic
Washington	1976	Mandatory	UBC
West Virginia	---	Legislation Defeated in 1987	None
Wisconsin	1914	Mandatory	Building Heat, Vent
Wyoming	1977	Mandatory Fire and Life Safety Only	UBC
Puerto Rico	1954	Mandatory for State Buildings Only	UBC
Virgin Islands	1948	Mandatory	VI Bldg. Code
District of Columbia	1987	Mandatory	BOCA

* Based Upon: "Directory of State Building Codes and Regulations", Fourth Edition, National Conference of States on Building Codes and Standards, Inc., Herndon, Virginia.

Table 5.2 (Continued)

Key

UBC	-	Uniform Building Code
SBC	-	Standard Building Code
Basic	-	Basic Building Code
NFPA 101	-	National Fire Protection Association Code for Safety to Life from Fire in Buildings and Structures
BOCA	-	BOCA National Building Code

5.4 Examples of Building Performance for Eastern United States Earthquakes

One of the unique features of ATC-14 is the extensive list of examples of building performance which are presented for each of the fifteen model building types. These sections were developed to provide the user with specific examples of documented building behavior which have occurred during past earthquakes. These examples helped to form the basis of the "performance characteristics" sections of the document and the specific "Statements" and "Concerns" of the evaluation procedure.

The examples were gathered from a large number of earthquake damage reports. These reports addressed the effects of earthquakes from the Western United States (such as San Francisco, 1906, Tehachapi, 1952, Alaska, 1964, San Fernando, 1971, etc.) and from around the world (such as Algeria, 1980, Argentina, 1977, Guatemala, 1976, Managua, 1972, Italy, 1976, etc.). These examples were typically taken from engineering reports written to describe the effects of these earthquakes.

None of the examples of building performance included in ATC-14 addressed the response of structures to Eastern United States earthquakes. This oversight occurred because the damaging earthquakes which have occurred in the Eastern United States are typically less well documented than their more recent Western counterparts. This lack of scientific information resulted because the earthquakes occurred in the distant past (Cape Ann, 1755, e.g.) and/or in a remote, sparsely populated area (New Madrid, 1811-1812, e.g.). The review panel indicated that there was enough information on some of the major eastern events to develop additional examples of building performance. Since such examples would provide a more complete listing of damaging earthquakes, a number were developed as part of this project. These examples are listed on the following pages.

5.4.1 Examples of Building Performance for Type 1 Wood Buildings-Dwellings (Page 86 of ATC-14)

17. Wood Frame One-Story Dwellings (general), Summerville, Charleston, 1886 (Dutton, C.E., 1890) pg. 275. Houses supported on 5 to 7 foot pillars of wood or brick and surrounded partially or wholly by a piazza also supported on pillars. Brick chimneys independently supported by arches or piers built up from ground. The whole building displaced one or two inches to the northward. The west end moved on the piers, while the east end carried the piers with it, leaving them inclined two inches from the vertical. All piers under the heavier portions of the house (particularly corner posts) were crushed at their summits, driven perceptibly into the ground and fissured obliquely, and several of them fell. Piers under the piazza were only slightly damaged and remained functional. Projections of both chimneys above the roof were thrown. Both crashed through the roof, one going through the floor to the ground. The basal portion of one chimney was crushed, intersected by oblique cracks and spread laterally five or six inches. The basal portion of the other was completely crushed and collapsed into conical heap. Wood pillars set at depth of two-to-three feet swung in all directions before returning to original positions, leaving annular space between posts and earth of one inch. Some of the smaller brick pillars which extended several inches into ground swung with the main building in like manner. Some were driven into earth with such force to produce surface depression for six inches to one foot in all directions from them. MM IX-X.

5.4.2 Examples of Building Performance for Type 2 Wood Buildings-Commercial or Industrial Buildings (Page 87 of ATC-14)

19. Northeastern Railroad Company Large Wooden Warehouses, Charleston, 1886 (Dutton, 1890). Structure about 400 feet long resting on piles. It was moved bodily a distance eight feet nine inches, causing one of its end to overhang its supports far enough for it to sag down two feet. It contained 1500 tons of freight at the time of the earthquake. MM IX-X.

20. New York and Charleston Warehouse and Navigation Large Wooden Warehouse, Charleston (Stocton, 1986). Building located on wharf built upon piles 60 feet long and capped with heavy timbers. Into these caps, heavy cypress supports are mortised, the tenons being 6 inches long, and upon these supports the building rests. It contained 45,000 tons of bulk storage. This enormous bulk was raised sufficiently to throw a very large number of tenons clear of the mortises, and the building being moved slightly, the tenons were unable to re-enter the mortises and rested on the caps. MM IX-X.

21. South Carolina Railroad Warehouses, Charleston (Dutton, 1890). A wharf 1,000 feet long on river side and 100 feet wide. Built on piles driven 40-60 feet. Solidly built with heavy timbers on piles. Wharf accommodated eight large warehouses built with sills resting on the wharf floor. All warehouses slid six to eight inches in one direction and from three to six inches in the perpendicular direction, without losing perpendicular of upright posts. However, nearly all hanging braces were torn from their sockets. Roofs undamaged. There was no sinking of piles. MM IX-X.

5.4.3 Examples of Building Performance for Unreinforced Masonry Bearing Wall Buildings (Page 244 of ATC-14)

26. College of Charleston Building, Charleston 1986 (Dutton, 1890). Central building constructed in 1828 to high standards, previous to the abandonment of shell lime. The wings were constructed later of recent and inferior masonry. The wings were badly shaken, requiring that they later be leveled. The central building, whose north and south walls were both forced outwards, had been substantially built. MM IX-X.

27. Unreinforced Masonry Buildings (general), Charleston, 1886 (Dutton, 1890). Ninety percent of brick buildings inspected were injured more or less. The extent of damage varied greatly, ranging from total demolition down to the loss of chimney tops and the dislodgement of plastering. The number of buildings completely demolished and leveled to the ground was not great. But there were several hundred which lost a large portion of their walls. Many left standing were so badly shattered that they were required to be pulled down. A majority, however, were repairable with earthquake rods and anchors. Bricks had "worked" in their embedding mortar and the mortar was disintegrated. The foundations were found to be badly shaken and their solidity greatly impaired. Many buildings had suffered horizontal displacement; vertical supports were out of plumb; floors out of level; joints parted in the wood work; beams and joists badly wrenched and in some cases dislodged from their sockets. The total estimated repair cost, including wood frame buildings as well, was estimated at 5-6 million dollars. MM IX-X.

28. Charleston Cotton Mills Building, Charleston, 1886 (Freeman, 1932). A well-built, five-story, brick factory building, standing on piles in very soft ground, withstood the earthquake with no damage except a few cracks in the brick tower beneath a 45-ton water tank and the tall brick chimney. The main building is 300 feet long by 98 feet wide with no stiffening partitions and no buttresses to the walls. This building was of "mill construction" and had brick bearing walls. The repair of all earthquake damage cost less than one-fourth of one percent of the value of the building. Many other well-built buildings escaped noteworthy damage. MM IX.

CHAPTER 6

REVISIONS TO THE SECTIONS WHICH ADDRESS THE SEISMIC EVALUATION OF BUILDINGS IN REGIONS OF LOW SEISMICITY

During the initial development of the ATC-14 document, the Project Engineering Panel felt that separate evaluation procedures should be developed for regions of low and high seismicity. They believed that the seismic evaluation procedure for buildings in regions of low seismicity should be less restrictive than that for regions of high seismicity; it would be sufficient to insure that there was a complete well balanced system for resisting the lateral loads, and that any falling hazards such as parapets, cornices, veneers, etc., were well anchored. As a result, two separate evaluation procedures were developed for each of the model buildings, with a shorter, less restrictive set of requirements for buildings in regions of low seismicity.

The panel members who reviewed the ATC-14 document for this NCEER sponsored project disagreed with the premise that the evaluation procedure for buildings in regions of low seismicity should be performed with procedures that are significantly less restrictive that those of the buildings in regions of high seismicity. They felt that a more specific and elaborate evaluation procedure was warranted since in addition to the general topics already addressed, they were also concerned with many of the issues which are included in the evaluation of buildings in regions of high seismicity. The lower seismic loading requirements would sufficiently distinguish the evaluation procedures. They also felt that since the engineers practicing in regions of low seismicity may not be as experienced in seismic design considerations, more specific and detailed direction would be necessary for these engineers to properly perform a seismic evaluation. As a result, the review panel recommended that a major expansion be made to the evaluation procedures for regions of low seismicity.

From the comments of the review panel, and the efforts of the entire project team, a large set of additions to the low seismicity evaluation procedures were developed. These additions included the following items:

1. Specific statements required for the evaluation of buildings in regions of high seismicity which were felt to also be applicable to areas of low seismicity.

2. Modification of specific statements required for the evaluation of buildings in regions of high seismicity to make them appropriate for areas of low seismicity.

3. New statements developed as a result of specific concerns expressed by members of the review panel.

4. Expanded introductory remarks.

Because of the volume of these proposed changes to the original ATC-14 document, it was decided that the most appropriate form of presentation would be to prepare completely new sections for the evaluations of buildings in regions of low seismicity. These proposed new sections are presented on the following pages. It is suggested that these new sections replace the corresponding sections in ATC-14 to provide a more complete seismic evaluation procedure for buildings in regions of low seismicity. Appendix C presents a set of checklists for these proposed new sections which would be useful in performing the preliminary field evaluation.

6.1 Wood-Frame Buildings (ATC-14 Section 5.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

5.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying potential seismic hazards.

5.5.1 Evaluation of Materials

Statement 5.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration. There is no substantial damage to wood elements due to bug infestation.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 5.5.2: There is no substantial damage to the wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical loads carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

5.5.2 Evaluation of Structural Elements

Statement 5.5.3: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by a design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 5.5.4: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 5.5.5: Walls with garage doors or other large openings are braced with plywood shear walls or are supported by adjacent construction through substantial positive ties.

Concern: All walls in wood-frame construction participate in the lateral system. When they have large openings, little or no resistance is available and they must be specially detailed or braced to other parts of the structure. Such bracing is not a conventional construction procedure. Lack of this bracing can lead to collapse of the wall.

Procedure: Evaluate wall Capacity/Demand ratio using the equivalent lateral force procedure. Check the ability of the walls and diaphragms to control open front displacements through torsional capacity, using the suggested special diaphragm analysis procedure in Section 4.4.6. Check that the diaphragm is a complete system with chords and collectors provided to deliver the lateral loads as required.

Recommended C/D Ratio: 1.0

Statement 5.5.6: All wall elements are bolted to the foundation sill at 6-foot spacing or less.

Concern: Buildings that are not bolted to the foundation may slide. If the building can fall a significant distance, this can lead to collapse in rare cases.

Procedure: Recommend that all wall elements be bolted to the foundation sill.

5.5.3 Evaluation of Foundations

Statement 5.5.7: There is positive connection of the posts to the foundation and the elements being supported.

Concern: The beams, posts, and foundation should be connected to prevent separation and loss of support.

Procedure: Report this condition to the owner and recommend that a positive connection be provided.

Statement 5.5.8: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 5.5.9: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w

Statement 5.5.10: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0

5.5.4 Evaluation of Non-Structural Elements

Statement 5.5.11: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If "government anchors" or corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0

Statement 5.5.12: The masonry chimney is tied at each floor and the roof.

Concern: Masonry chimneys can collapse if they are not tied to the buildings at each floor level.

Procedure: Verify that the chimney is constrained by the structural elements. If it can fall, then recommend that the chimney be tied at each floor.

Recommended C/D Ratio: 1.0

6.2 Steel Moment Resisting Frame Buildings (ATC-14 Section 6.1.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

6.1.5 Evaluation of Buildings In Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading needs to consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

6.1.5.1 Evaluation of Materials

Statement 6.1.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 6.1.5.2: There is no substantial damage to the wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical load carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.1.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Statement 6.1.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.1.5.2 Evaluation of Structural Elements

Statement 6.1.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 6.1.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 6.1.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effect of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.1.5.8: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 6.1.5.9: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e. a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic response at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 6.1.5.10: There is no immediately adjacent structure that has floors/levels that do not match those of the building being evaluated. A neighboring structure will be considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated.

Concern: Moment frame buildings that are immediately adjacent to buildings that have different story heights are subject to pounding. The roof diaphragm of the adjacent building could pound into the exterior wall columns, leading to column distress and possible local collapse.

Procedure: Recommend the addition of floor-to-floor elements that will minimize the effects of pounding damage where it occurs.

Statement 6.1.5.11: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the strength of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 6.1.5.12: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include reentrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

6.1.5.3 Evaluation of Foundations

Statement 6.1.5.13: The columns in the lateral force resisting frames are substantially anchored to the building foundation.

Concern: The anchorage of the frame columns to the foundation is a part of the lateral load resisting path that may not have been designed to have adequate shear or tension capacity.

Procedure: Determine column base forces from an equivalent lateral force procedure to estimate the requirements for tension and/or shear reinforcement.

Recommended C/D Ratio: 1.0.

Statement 6.1.5.14: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 6.1.5.15: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.1.5.16: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.1.5.4 Evaluation of Non-Structural Elements

Statement 6.1.5.17: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 6.1.5.18: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.3 Braced Steel Frame Buildings (ATC-14 Section 6.2.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

6.2.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading needs to consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover.the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

6.2.5.1 Evaluation of Materials

Statement 6.2.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system and recommend that corrective action be taken. If analyses of existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 6.2.5.2: There is no substantial damage to the wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical load carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.2.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Statement 6.2.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.2.5.2 Evaluation of Structural Elements

Statement 6.2.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is continuouss load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 6.2.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 6.2.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.2.5.8: The lateral force resisting elements form a well-balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using 3D procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 6.2.5.9: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e., a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic activity at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 6.2.5.10: All the brace connections are able to develop the yield capacity of the diagonals.

Concern: Failure of connections is generally not a ductile mode of failure. It is more desirable to have any inelastic action occur in the members rather than the connections.

Procedure: Check the connection strength against the demand created by an equivalent lateral force procedure.

Recommended C/D Ratio: 0.4 R_w or a value for which the connection strength is greater than the tensile capacity of the braces, whichever is less.

Statement 6.2.5.11: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 6.2.5.12: There is special diaphragm reinforcing at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include reentrant corners may cause the wings of the structure to attempt to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

6.2.5.3 Evaluation of Foundations

Statement 6.2.5.13: The columns in the lateral force resisting frames are substantially anchored to the building foundation.

Concern: The anchorage of the frame columns to the foundation is a part of the lateral load resisting path that may not have been designed to have adequate shear or tension capacity.

Procedure: Determine column base forces from an equivalent lateral force procedure to estimate the requirements for tension and/or shear reinforcement.

Recommended C/D Ratios: 0.2 R_w for a shear friction type transfer or for expansion anchors, or 1.0 otherwise.

Statement 6.2.5.14: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 6.2.5.15: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.2.5.16: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.2.5.4 Evaluation of Non-Structural Elements

Statement 6.2.5.17: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 6.2.5.18: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12′-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.4 Light Steel Moment Frame Buildings with Longitudinal Tension-Only Bracing (ATC-14 Section 6.3.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

6.3.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

6.3.5.1 Evaluation of Materials

Statement 6.3.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to a more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 6.3.5.2: There is no substantial damage to the wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical load carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.3.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.3.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.3.5.2 Evaluation of Structural Elements

Statement 6.3.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 6.3.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building system does not incorporate redundancy, recommend that additional lateral force resisting elements be added.

Statement 6.3.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft areas, such as braced frames with open bays at the base, or other severe vertical strength irregularities can cause concentration of inelastic response, interstory drift, and non-structural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses. If a soft story exists that cannot be justified through sufficient capacity (see Recommended C/D below), recommend that new lateral force resisting elements be added to eliminate the discontinuities.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.3.5.8: The lateral force resisting elements form a well-balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using 3D procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 6.3.5.9: All light metal roof panels are connected to the roof framing at 12 inch centers.

Concern: The lack of connection between the roof panels and the framing elements creates a falling hazard. The lack of adequate connection may also cause improper diaphragm action.

Procedure: Report this condition to the owner and recommend that corrective action be taken.

Statement 6.3.5.10: All wall panels (metal, fiberglass, or cement asbestos) are connected to the framing.

Concern: Without proper connection of the wall panels to the framing, these panels can present a falling hazard.

Procedure: Recommend that all panels be positively connected.

6.3.6.4 Evaluation of Structural Details

Statement 6.3.5.11: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 6.3.5.12: There is special diaphragm reinforcing at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include re-entrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

6.3.5.3. Evaluation of Foundations

Statement 6.3.5.13: The columns in the lateral force resisting frames are substantially anchored to the building foundation.

Concern: The anchorage of the frame columns to the foundation is a part of the lateral load resisting path that may not have been designed to have adequate shear or tension capacity.

Procedure: Determine column base forces from an equivalent lateral force procedure to estimate the requirements for tension and/or shear reinforcement.

Recommended C/D Ratios: 0.2 R_w for friction or expansion anchors, or 1.0 otherwise.

Statement 6.3.5.14: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 6.3.5.15: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.3.5.16: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.3.5.4 Evaluation of Non-Structural Elements

Statement 6.3.5.17: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 6.3.5.18: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.5 Steel Frame Buildings with Cast-in-Place Concrete Walls (ATC-14 Section 6.4.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

6.4.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding, and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

6.4.5.1 Rapid Evaluation for Shear Stress in Concrete Walls

Concern: Concrete shear wall buildings should be provided with an amount of wall area that will result in shear capacity that is sufficient to resist the lateral forces. A quick estimation of the shear stress on the concrete walls should be performed in all evaluations of this building type in regions of high or moderate seismicity.

Procedure: Generate the lateral loads using the rapid evaluation procedure presented in Section 4.4.2, checking the first floor level, and all other levels that could also be subjected to high shear stresses. Estimate the average wall shear stress, v_{AVG}, using the following formula:

$$v_{AVG} = V_j/A_w$$

where: V_j = Story shear at the level under consideration determined from the loads generated by the rapid evaluation procedure.

A_w = Summation of the horizontal cross sectional area of all shear walls in the direction of loading with height to width ratios less than 2. The wall area should be reduced by the area of any openings.

If v_{AVG} is greater than 60 psi, a more detailed evaluation of the structure should be performed. This evaluation should employ a more accurate estimation of the level and distribution of the lateral loads by using the procedures suggested in Section 4.4. Calculate the wall capacities using the provisions of Chapter 26 of the Uniform Building Code (ICBO, 1985), and compute Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

6.4.5.2 Evaluation of Materials

Statement 6.4.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 6.4.5.2: There is no substantial damage to the wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical load carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.4.5.3 Evaluation of Structural Elements

Statement 6.4.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommended that additional lateral force resisting elements be added.

Statement 6.4.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.4.5.8: The reinforcing steel for concrete walls is greater than .0025 times the gross area of the wall along both the longitudinal and transverse axes, at a spacing that does not exceed 18 inches.

Concern: A minimum amount of steel reinforcing is required for concrete walls to provide acceptable inelastic performance.

Procedure: Calculate the capacity of the walls with the reinforcing that is provided. Compute Capacity/Demand ratios that result from use of the equivalent lateral force procedure.

Recommended C/D Ratio: 0.2 R_w.

Statement 6.4.5.9: All metal deck floors and roofs have a reinforced concrete topping slab with a minimum thickness of 3 inches.

Concern: Metal deck diaphragms without topping slabs may not have sufficient strength.

Procedure: Use an equivalent lateral force procedure to calculate a Capacity/Demand ratio for the strength of the bare metal deck diaphragm elements. The demand from this analysis should be compared with the minimum requirements for diaphragms given in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.10: The lateral force resisting elements form a well-balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections which may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using 3D procedures which are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 6.4.5.11: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e. a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic response at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.12: There is reinforcing in each diaphragm to transfer loads to the shear walls.

Concern: Shear walls are effective only as long as they are sufficiently connected to the diaphragm. The connection can be by shear along the interface or collector bars embedded in the wall.

Procedure: Determine the equivalent lateral force demand on the diaphragm and verify the adequacy of the available diaphragm reinforcing by calculating Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.13: All walls are continuous to the foundation.

Concern: Discontinuous walls can lead to column shear or axial load failures at the base of the discontinuous wall. Column failures can lead to fall or partial collapse.

Procedure: Compare the column shear, moment, and axial force capacity at the discontinuity to the demands generated by the equivalent lateral force procedure. Check the diaphragm capacity to transfer these loads to other vertical elements. Check the story stiffness to be sure that no soft story condition exists.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.4.5.14: There is positive connection between the shear walls and the steel beams and columns.

Concern: Substantial shear transfer between the structural steel and the concrete must occur for the shear walls to be fully effective. Especially important is the connection to the column for overturning forces. The connections should include welded studs, welded reinforcing steel, or fully encased steel elements with longitudinal reinforcing and ties.

Procedure: Calculate the effective overturning demand for the walls and determine the Capacity/Demand ratios for the shear transfer to the steel elements using the equivalent lateral force procedure. A value for shear friction between steel and concrete should be included only if the steel element is completely encased with reinforced concrete.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.15: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.16: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include reentrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.17: The diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length, and the available length appears sufficient.

Concern: Shear wall buildings are dependent on diaphragms for proper performance. Substantial openings next to walls can prevent the proper transfer of load between the walls and the diaphragms.

Procedure: Verify that there is sufficient strength to deliver the appropriate amount of lateral load to the shear wall using the equivalent lateral force procedure to calculate Capacity/Demand ratios for the load transfer.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.18: There is special wall reinforcement placed around all openings.

Concern: If the openings are not properly reinforced, they can reduce the strength of the walls. This can lead to degradation of the wall around the openings.

Procedure: Determine the capacity of the spandrels and piers considering all available reinforcing steel that crosses the critical sections. Calculate and evaluate Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.19: The stirrups in all coupling beams over means of egress are spaced at 8 d_b or less and are anchored into the core with hooks of 135 degrees or more.

Concern: Earthquake damage in coupled shear wall buildings typically includes debris from lightly reinforced spandrel beams that may fall and block means of egress.

Procedure: Use the equivalent lateral force procedure to calculate the capacity of coupling beams and determine Capacity/Demand ratios.

Recommended C/D Ratio: 0.2 R_w.

6.4.5.4 Evaluation of Foundations

Statement 6.4.5.20: All vertical wall reinforcing is doweled into the foundation.

Concern: The lack of sufficient dowels will create a weak plane that may not have adequate shear or tension capacity.

Procedure: Determine the dowel requirements from the ACl 318 minimum value or the actual values from an analysis using the equivalent lateral force procedure. Calculate Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.21: The shear wall columns are substantially anchored to the building foundation.

Concern: The anchorage of the shear wall columns to the foundation is a part of the lateral load resisting path that may not have been designed to have adequate shear or tension capacity.

Procedure: Determine column base forces from an equivalent lateral force procedure to determine the requirements for tension and/or shear reinforcement.

Recommended C/D Ratio: 0.2 R_w for friction or expansion anchors, or 1.0 otherwise.

Statement 6.4.5.22: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 6.4.5.23: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.4.5.24: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.4.5.25: For buildings taller than six stories in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions (M > 5.5 high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

6.4.5.5 Evaluation of Non-Structural Elements

Statement 6.4.5.26: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0

Statement 6.4.5.27: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0

6.6 Steel Frame Buildings with Infilled Walls of Unreinforced Masonry (ATC-14 Section 6.5.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

6.5.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

6.5.5.1 Evaluation of Materials

Statement 6.5.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 6.5.5.2: The mortar cannot be scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar.

Concern: Mortar that is severely eroded or can easily be scraped away has been found to have low shear strength, which also results in low wall strengths. Testing procedures are required to determine the in-plane shear strength and adequacy of the walls. Inform the owner that eroded areas should be repaired.

Procedure: Perform the wall tests to establish the capacity of the walls. Use an equivalent lateral force procedure to calculate Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 6.5.5.3: There is no substantial damage to the wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical load carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.5.5.4: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.5.5.5: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

6.5.5.2 Evaluation of Structural Elements

Statement 6.5.5.6: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 6.5.5.7: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 6.5.5.8: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 6.5.5.9: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

Concern: Heavy exterior walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during seismic response. If these walls are bearing walls, partial floor collapse may result. The hazard created increases with the height above the building base. The amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, and the type of soil.

Procedure: Calculate the inertial weight tributary to the anchorage level as the mass within one-half the distance between adjacent levels of anchorage. Use Equation 4.12 and Table 4.8 to estimate the lateral force on this anchorage. If "government anchors" are used for the wall anchorage, a testing program to determine this capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 4.0 for buildings with wood diaphragms (ABK, 1984); otherwise, 1.0.

Statement 6.5.5.10: The steel frames form a complete vertical load carrying system.

Concern: This building type has exhibited generally acceptable performance because it contains a complete semi-ductile steel vertical frame system that interacts favorably with the masonry infills. If any of the masonry walls carry significant gravity load, the floors may be subject to partial collapse as the walls crack, deteriorate, and loose their vertical load carrying ability. Otherwise, for the steel frame under yield level loads, the walls continue to resist lateral loads and dissipate energy while the steel frame supports the gravity loads.

Procedure: Evaluate the walls as if they were in an unreinforced masonry bearing wall building, using the procedures of Chapter 10.

Statement 6.5.5.11: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion is taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using 3D procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 6.5.5.12: The infilled walls are continuous to the base of the building.

Concern: Discontinuous infilled walls can lead to soft stories that cause the drift and energy dissipation to focus in specific areas. This can lead to amplification of local demands that could result in a concentration of inelastic response, interstory drift, nonstructural damage, and even collapse.

Procedure: Use the equivalent lateral force procedure to evaluate the distribution of loads at the wall discontinuity. Check if redistribution of force to other vertical lateral force resisting elements can occur.

Recommended C/D Ratio: 0.4 R_w of the lateral load carrying elements below the infill if no redistribution to other walls can occur; 1.0 if the lateral loads can be redistributed.

Statement 6.5.5.13: For buildings founded on soft soils (S_3 and S_4), the height/thickness ratios of the infilled wall panels in a one-story building are less than 14.

Concern: The dynamic stability of unreinforced masonry wall panels depends on their height/thickness ratios. In regions of low seismicity, dynamic stability should be adequate except for walls with high H/T ratios in buildings founded on moderately soft soils (ABK, 1984A). This stability is also dependent on the response of the floor and roof diaphragms. If the building has cross walls or concrete diaphragms, the allowable height/thickness ratios can be increased to 18.

Procedure: Calculate the out-of-plane demand using Section 4.4.5 and the capacity of the wall.

Recommend C/D Ratio: 3.

Statement 6.5.5.14: For buildings founded on soft soils (S_3 and S_4), the height/thickness ratios of the top story infilled wall panels in a multi-story building are less than 9.

Concern: The dynamic stability of unreinforced masonry wall panels depends on their height/thickness ratios. In regions of low seismicity, dynamic stability should be adequate except for walls with high H/T ratios in buildings founded on moderately soft soils (ABK, 1984A). This stability is also dependent on the response of the floor and roof diaphragms. If the building has cross walls or concrete diaphragms, the allowable height/thickness ratios can be increased to 14.

Procedure: Calculate the out-of-plane demand using Section 4.4.5 and the capacity of the wall.

Recommended C/D Ratio: 3.

Statement 6.5.5.15: For buildings founded on soft soils (S_3 and S_4), the height/thickness ratios of the infilled wall panels in other stories of a multi-story building are less than 20.

Concern: The dynamic stability of unreinforced masonry wall panels depends on their height/thickness ratios. In regions of low seismicity, dynamic stability should be adequate except for walls with high H/T ratios in buildings founded on moderately soft soils (ABK, 1984A). This stability is also dependent on the response of the floor and roof diaphragms.

Procedure: Calculate the out-of-plane demand using Section 4.4.5 and the capacity of the wall.

Recommended C/D Ratio: 3.

Statement 6.5.5.16: All infilled panels are constructed to encompass the steel frames around their entire perimeter.

Concern: In order to perform properly, the masonry infill must contact the steel framing elements on all four sides. Without proper attachment, the infill may not be able to provide the expected performance, and also may be subject to out-of-plane failure. This condition sometimes occurs when clerestory windows are provided at the top of the infilled panels.

Procedure: Recommend that positive connection between the infill and the frame be added.

Statement 6.5.5.17: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.6.

Recommended C/D Ratio: 1.0.

Statement 6.5.5.18: No clay-tile arch floors are present.

Concern: Clay-tile arch floor systems are heavy, brittle elements, whose seismic behavior is not well understood. Damage due to in-plane movements and vertical acceleration creates the potential for materials to fall from the slab underside. Solid brick arches are not of concern.

Procedure: Where clay-tile arch floors exist, perform analyses for damage potential due to in-plane motion, using conservative values for allowable stresses. Evaluate the potential for damage to cause materials to fall from the slab underside.

Recommended C/D Ratio: 4.0.

6.5.5.3 Evaluation of Foundations

Statement 6.5.5.19: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 6.5.5.20: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_{W}.

Statement 6.5.5.21: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 6.5.5.22: For buildings taller than six stories in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions (M > 5.5 high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

6.5.5.4 Evaluation of Non-Structural Elements

Statement 6.5.5.23: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 6.5.5.24: All exterior cladding, veneer courses, and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.7 Moment Resisting Cast-in-Place Concrete Buildings (ATC-14 Section 7.1.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

7.1.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq .10$ g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

7.1.5.1 Rapid Evaluation of Reinforced Columns

Concern: Reinforced concrete frame buildings have sometimes proven to present a life-safety hazard in past earthquakes because of inadequate column shear capacity. A quick estimation of the shear stress in the concrete frame columns should be performed in all evaluations of this building type in regions of high or moderate seismicity.

Procedure: Generate the loads using the rapid evaluation procedure presented in Section 4.4.2, checking the first floor level and all other levels where the columns could be subjected to high shear stresses. Estimate the average column shear stress, v_{AVG}, as follows:

$$v_{AVG} = \frac{n_c}{n_c - n_f} \quad \frac{V_j}{A_c}$$

where: n_c = Total number of columns
n_f = Total number of frames in the direction of loading
V_j = Story shear at the level under consideration, determined from the loads generated by the rapid evaluation procedure
A_c = Summation of the cross sectional area of all columns in the story under consideration

If the average column shear stress is greater than 60 psi, a more detailed evaluation of the structure should be performed. This evaluation should employ a more accurate estimation of the level and distribution of the lateral loads by using the procedures suggested in Section 4.4. Calculate the column shear capacities using the provisions of Chapter 26 of the Uniform Building Code (ICBO, 1985) and compute Capacity/Demand ratios.

Recommended C/D Ratio: 1.0. Many of the concerns in the following statements will address the details necessary to provide ductile column behavior.

7.1.5.2 Rapid Estimation of Story Drift (All buildings).

Concern: Moment resisting frame structures sre typically not as stiff as similar shear wall or braced frame buildings. This flexibility can result in large interstory drift, which may lead to extensive nonstructural damage.

Procedure: Use the following formula with the loads generated by the rapid evaluation procedure to estimate the story drift, Q, at any level:

$$\Delta = \frac{k_b + k_c}{k_b k_c} \quad \frac{h}{4500} \quad V_c$$

where: k_b = (I/L) Beam
k_c = (I/L) Column
h = Story height, inches
I = Moment of inertia, in^4
L = Center to center length, inches
V_c = Average shear in each column.

Calculate this value from the rapid evaluation procedure given in Section 4.4.2. If the estimated drift exceeds 0.005 at any story level, the structure should be evaluated using full-frame analysis using the force level and the anticipated distribution of lateral forces to the moment resisting frames using the recommendations of Section 4.4. Note that the V_c value used for the rapid drift estimation should be calculated considering the relative rigidities of frame elements.

7.1.5.2 Rapid Evaluation of Story Drift

Statement 7.1.5.1: *The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.*

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycless can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where the deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 7.1.5.2: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.3: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

7.1.5.4 Evaluation of Structural Elements

Statement 7.1.5.4: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of low seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.5: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 7.1.5.6: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 7.1.5.7: The shear capacity of the frame columns is greater than the moment capacity.

Concern: Shear failure of columns tend to be brittle and can lead to collapse. The ultimate shear capacity should be checked against the ultimate moment capacity.

Procedure: Use the rapid analysis procedure outlined in Section 4.4.2 for regions of high seismicity to check the shear capacity and moment capacity of the columns. If column shear failures are indicated, use an equivalent lateral force procedure to evaluate C/D ratios for the column elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 7.1.5.8: There are no infills of concrete or masonry placed in the concrete frames that are not isolated from the structural elements.

Concern: Infilled walls used for partitions or walls around the stair or elevator towers that are not adequately isolated will alter the seismic response of the structure. Evaluation of considcrations for frame structures will therefore be inappropriate.

Procedure: Evaluate the building as an infilled wall structure using the procedures of Section 7.3.

Statement 7.1.5.9: The lateral force resisting elements form a well-distributed and balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities may cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 7.1.5.10: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e. a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear which can be significantly different from that of regular buildings. This can lead to a concentration of inelastic activity at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.11: All of the frames continue to the building base.

Concern: All of the frames carry shear and overturning forces. Any frames that do not continue to the foundation must deliver their shear and overturning to other structural elements. Unless there are supplementary elements specifically detailed to take these loads, these elements may not have sufficient capacity.

Procedure: Evaluate the demands on the supporting elements using the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.12: The moment capacity of the columns appears to be greater than that of the beams.

Concern: Extensive column hinging may lead to extensive column damage and possibly loss of axial capacity. The inelastic activity should be moment yielding of the beam elements.

Procedure: Compare the summation of the beam moment capacities including slab width to the summation of column moment capacities. The columns should be 20 percent stronger than the beams to ensure proper action.

Statement 7.1.5.13: All metal deck floors and roofs have a reinforced concrete topping slab with a minimum thickness of 3 inches.

Concern: Metal deck diaphragms without topping slabs may not have sufficient strength.

Procedure: Use an equivalent lateral force procedure to calculate a Capacity/Demand ratio for the strength of the bare metal deck diaphragm elements. The demand from the analysis should be compared with the minimum requirements for diaphragms given in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.14: There is no immediately adjacent structure having floors/levels that do not match those of the building being evaluated. A neighboring structure will be considered to be "immediately adjacent" if it is within 2 inches times the number of stories awgy from the building being evaluated.

Concern: Moment frame buildings immediately adjacent to shorter buildings that have different story heights are subject to pounding. The roof diaphragm of the shorter adjacent building could pound into the exterior wall columns, leading to column distress and possible local collapse.

Procedure: Recommend the addition of floor-to-floor elements that will minimize the effects of pounding where it occurs.

Statement 7.1.5.15: Frame columns have ties spaced at d or less throughout their length, and at 8 d_b or d/2 at all potential plastic hinge locations.

Concern: Non-ductile shear failures may occur for columns with widely spaced ties. Without closely spaced ties, the columns may also be unable to maintain the yield level moments under repeated cycles.

Procedure: Calculate the maximum shear force that can be generated in the columns by analyzing the column moment capacity under maximum axial load. Compute the maximum axial load as 1.4 times the summation of the dead, live, and seismic forces. Calculate Capacity/Demand ratios for the shear in the columns at the maximum shear force.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.16: All column bar lap splice lengths are greater than 30 d_b long, and are enclosed by ties spaced at $8d_b$ or less.

Concern: Splices of inadequate length may lead to column distress and even failure. This problem will be amplified by spalling of concrete cover that could occur during large drifts.

Procedure: Compare the splice length provided with that required by the ACI requirements (ACI, 1983, Sections 12.2 and 12.15), as appropriate. Calculate demand using the equivalent lateral force procedure.

Recommended C/D Ratio: 0.2 R_w.

Statement 7.1.5.17: The positive moment strength at the face of the joint is greater than 1/3 of the negative moment strength. At least 20 percent of the steel provided at the joints for either positive or negative moment is continuous throughout the member.

Concern: Yield level moments require reinforcing steel between the point of inflection and the support because the seismic moments can be much greater than the gravity load moments. Continuous slab reinforcement adjacent to the beam may be considered as continuous top reinforcement.

Procedure: Evaluate the moment demands using the equivalent lateral force procedure. Compare these moments to capacity based on ACI requirements, by calculating Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.18: All beams have stirrups spaced at d/2 or less throughout their length, and at 8 d_b or d/4 at potential hinge locations.

Concern: Without closely spaced stirrups, the beams may be unable to maintain the yield level moments under repeated cycles.

Procedure: Determine the beam shear demands using the equivalent lateral force procedure. For calculation of shear capacity, use only reinforcement that is effective for shear reversals.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.19: Bent-up longitudinal steel is not used for shear reinforcement.

Concern: Bent up shear reinforcement is not adequate under reversing moments.

Procedure: Evaluate the beam shear demands using the equivalent lateral force procedure. Por calculation of shear capacity, use only reinforcement that is effective for shear reversals.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.20: Column ties extend through all exterior beam-column joints with their typical spacing.

Concern: Unreinforced exterior beam-column joints may not be able to develop the strength of the connected members. This can lead to joint yielding.

Procedure: Compare joint capacity with the shear created by the summation of the beam yield moments.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.21: There is significant tensile capacity at reentrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include re-entrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.22: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provision for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

7.1.5.5 Evaluation of Foundations

Statement 7.1.5.23: All longitudinal column steel is doweled into the foundation.

Concern: The lack of sufficient dowels creates a weak plane that may not have adequate shear or tension capacity, especially for overturning forces.

Procedure: Determine the dowel requirements from the ACI 318 minimum value (ACI, 1983) or the actual values from an analysis using the equivalent lateral force procedure, and calculate C/D ratios.

Recommended C/D Ratio: 1.0.

Statement 7.1.5.24: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 7.1.5.25: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 $R_{w.}$

Statement 7.1.5.26: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

7.1.5.6 Evaluation of Non-Structural Elements

Statement 7.1.5.27: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recomended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 7.1.5.28: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard crented increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.8 Cast-in-Place Concrete Shear Wall Buildings (ATC-14 Section 7.2.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

7.2.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake trading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features, During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

7.2.5.1 Rapid Evaluation of Shear Stress in Concrete Walls

Concern: Concrete shear wall buildings should be provided with an amount of wall area that will result in shear capacity that is sufficient to resist the lateral forces. A quick estimation of the shear stress on the concrete walls should be performed in all evaluations of this building type in regions of high or moderate seismicity.

Procedure: Generate the lateral loads using the rapid evaluation procedure presented in Section 4.4.2, checking the first floor level, and all other levels that could also be subjected to high shear stresses. Estimate the average wall shear stress, V_{AVG}, using the following formula:

$$V_{AVG} = V_j/A_w$$

where: V_j = Story shear at the level under consideration determined from the loads generated by the rapid evaluation procedure

Aw = Summation of the horizontal cross sectional area of all shear walls in the direction of loading with height-to width ratios less than 2. The wall area should be re duced by the area of any openings.

If V_{AVG} is greater than 60 psi, a more detailed evaluation of the structure should be performed. This evaluation should employ a more accurate estimation of the level and distribution of the lateral loads by using the procedures suggested in Section 4.4. Calculate the wall capacities using the provisions of Chaper 26 of the UBC (ICBO, 1985), and compute Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

7.2.5.2 Evaluation of Materials

Statement 7.2.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 7.2.5.2: There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil loop anchors have not been used.

Concern: Corrosion in post-tensioned anchorages can lead to the release of post-tensioning during ground shaking and cause failure of the lateral force resisting system. Coil loop anchors, which may be susceptible to failure, have been prohibited by current standards.

Procedure: Inspect a sample of the concrete in the area of the post-tensioning anchorage to determine its condition. Determine the extent and cause of the deterioration. Recommend that specific corrective action be taken.

Statement 7.2.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

7.2.5.3 Evaluation of Structural Elements

Statement 7.2.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 7.2.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: $0.4\ R_w$.

Statement 7.2.5.8: The reinforcing steel for concrete walls is greater than.0025 times the gross area of the wall along both the longitudinal and transverse axes, at a spacing that does not exceed 18 inches.

Concern: A minimum amount of steel reinforcing is required for concrete walls to provide acceptable inelastic performance.

Procedure: Calculate the capacity of the walls with the reinforcing that is provided. Compute Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: $0.2\ R_w$.

Statement 7.2.5.9: All metal deck floors and roofs have a reinforced concrete topping slab with a minimum thickness of 3 inches.

Concern: Metal deck diaphragms without topping slabs may not have sufficient strength.

Procedure: Use an equivalent lateral force procedure to calculate a Capacity/Demand ratio for the strength of the bare metal deck diaphragm elements. The demand from this analysis should be compared with the minimum requirements for diaphragms given in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.10: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse. Note that these buildings typically have better diaphragms and should have more inherent strength than steel buildings.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 7.2.5.11: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e., a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic response at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.12: All walls are continuous to the foundation.

Concern: Discontinuous walls can lead to column shear or axial load failures at the base of the discontinuous wall. Column failures can lead to fall or partial collapse.

Procedure: Compare the column shear, moment, and axial force capacity at the discontinuity to the demands generated by the equivalent lateral force procedure. Check the diaphragm capacity to transfer these loads to other vertical elements. Check the story stiffness to be sure that no soft story condition exists.

Recomme ded C/D Ratio: 0.4 R_w.

Statement 7.2.5.13: There is reinforcing in each diaphragm to transfer loads to the shear walls.

Concern: Shear walls are effective only as long as they are sufficiently connected to the diaphragm. The connection can be by shear along the interface or collector bars embedded in the wall.

Procedure: Determine the equivalent lateral force demand on the diaphragm and verify the adequacy of the available diaphragm reinforcing by calculating Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.14: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.15: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include re entrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.16: The diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length.

Concern: Shear wall buildings are dependent on diaphragms for proper performance. Substantial openings next to walls can prevent the proper transfer of load between the walls and the diaphragms.

Procedure: Verify that there is sufficient strength to deliver the appropriate amount of lateral load to the shear wall using the equivalent lateral force procedure to calculate Capacity/Demand ratios for the load transfer.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.17: There is special wall reinforcement placed around all openings.

Concern: If the openings are not properly reinforced, they can reduce the strength of the walls. This can lead to degradation of the wall around the openings.

Procedure: Determine the capacity of the spandrels and piers considering all available reinforcing steel that crosses the critical sections. Calculate and evaluate Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

7.2.5.4 Evaluation of Foundations

Statement 7.2.5.18: All vertical wall reinforcing is doweled into the foundation.

Concern: The lack of sufficient dowels creates a weak plane that may not have adequate shear or tension capacity

Procedure: Determine the dowel requirements as the maximum of the ACI 318 minimum value (ACI, 1983) or the actual values from an analysis using the equivalent lateral force procedure, and calculate Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.19: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 7.2.5.20: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 7.2.5.21: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 7.2.5.22: For buildings taller than six stories in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions (M > 5.5 high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

7.2.5.5 Evaluation of Non-Structural Elements

Statement 7.2.5.23: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 7.2.5.24: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.9 Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry (ATC-14 Section 7.3.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

7.3.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$ .10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

7.3.5.1 Evaluation of Materials

Statement 7.3.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where the deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 7.3.5.2: The mortar cannot be scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar.

Concern: Mortar that is severely eroded or can easily be scraped away has been found to have low shear strength, which also results in low wall strengths. Testing procedures are required to determine the in-plane shear strength and adequacy of the walls. Inform the owner that eroded areas should be repaired.

Procedure: Perform the wall tests to estimate the capacity of the walls. Use an equivalent lateral force procedure to calculate Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 7.3.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Statement 7.3.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

7.3.5.2 Evaluation of Structural Elements

Statement 7.3.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 7.3.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 7.3.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 7.3.5.8: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

Concern: Heavy exterior walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during seismic response. If these walls are bearing walls, partial floor collapse may result. The hazard created increases with the height above the building base. The amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, and the type of soil.

Procedure: Calculate the inertial weight tributary to the anchorage level as the mass within one-half the distance between adjacent levels of anchorage. Use Equation 4.12 and Table 4.8 to estimate the lateral force on this anchorage. If "government anchors" are used for the wall anchorage, a testing program to determine this capacity is recommended, Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 4.0 for buildings with wood diaphragms (ABK, 1984); otherwise, 1.0.

Statement 7.3.5.9: The infilled walls are continuous to the soffits of the frame beams.

Concern: Unreinforced masonry infilled walls that stop below the beam soffits create a "short column" condition, which may produce large loads in the columns and possibly cause a brittle shear failure. This condition is seen in damaged buildings after nearly every large earthquake and could lead to collapse.

Procedure: Evaluate the shear forces that occur in the "short" columns at the openings using an equivalent lateral force procedure.

Recommended C/D Ratio: $0.4R_w$.

Statement 7.3.5.10: The concrete frames form a complete vertical load carrying system.

Concern: This building type can exhibit acceptable performance if it contains a complete concrete vertical frame system that interacts favorably with the masonry infills. If any of the masonry walls carry significant gravity load, the floors may be subject to partial collapse. Otherwise, under yield level loads, the walls continue to resist lateral loads and dissipate energy while the concrete frame supports the gravity loads.

Procedure: Evaluate the walls as if they were in an unreinforced masonry bearing wall building, using the procedures of Chapter 10.

Statement 7.3.5.11: The lateral force resisting elements form a well-balanced system that is not subject to significant torsion.

Concern: Plan irregularities and/or soft stories can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use $1.4\ R_w$ times maximum calculated drift for evaluation.

Statement 7.3.5.12: The infilled walls are continuous to the base of the building.

Concern: Discontinuous infilled walls can lead to soft stories that cause the drift and energy dissipation to focus in specific areas. This can lead to amplification of local demands that could result in a concentration of inelastic response, interstory drift, nonstructural damage, and even collapse.

Procedure: Use the equivalent lateral force procedure to evaluate the distribution of loads at the wall discontinuity. Check if redistribution of force to other vertical lateral force resisting elements can occur.

Recommended C/D Ratio: 0.4 R_w if no redistribution can occur, 1.0 if the lateral loads can be redistributed.

Statement 7.3.5.13: For buildings founded on soft soils (S_3 and S_4), the height/thickness ratios of the infilled wall panels in a one-story building are less than 14.

Concern: The dynamic stability of unreinforced masonry wall panels depends on their height/thickness ratio. This stability is also dependent on the response of the floor and roof diaphragms. If the building has crosswalls or concrete diaphragms, the allowable height/thickness ratios can be increased to 18.

Procedure: Calculate the out-of-plane demand using Section 4.4.5 and the capacity of the wall.

Recommended C/D Ratio: 3.

Statement 7.3.5.14: For buildings founded on soft soil (S_3 and S_4), the height/thickness ratios of the top story infilled wall panels in a multi-story building are less than 9.

Concern: The dynamic stability of unreinforced masonry wall panels depends on their height/thickness ratio. In regions of low seismicity, dynamic stability should be adequate except for walls with high H/T ratios in buildings founded on moderately soft soils (ABK, 1984). This stability is also dependent on the response of the floor and roof diaphragms. If the building has crosswalls or concrete diaphragms, the allowable height/thickness ratios can be increased to 14.

Procedure: Calculate the out-of-plane demand using Section 4.4.5 and the capacity of the wall.

Recommended C/D Ratio: 3.

Statement 7.3.5.15: For buildings founded on soft soils (S_3 and S_4), the height/thickness ratios of the infill wall panels in other stories of a multi-story building are less than 20.

Concern: The dynamic stability of unreinforced masonry wall panels depends on their height/thickness ratio. In regions of low seismicity, dynamic stability should be adequate except for walls with high H/T ratios in buildings founded on moderately soft soils (ABK, 1984). This stability is also dependent on the response of the floor and roof diaphragms.

Procedure: Calculate the out-of-plane demand using Section 4.4.5 and the capacity of the wall.

Recommended C/D Ratio: 3.

Statement 7.3.5.16: The infilled walls are not of cavity construction, which results in a situation where the exterior and interior courses are not well bonded.

Concern: Insufficient perpendicular-to-wall strength can lead to exterior course spalling or out-of-plane wall failure.

Procedure: Recommend that out of plane bracing be added.

Statement 7.3.5.17: All infilled panels are anchored to or encompassed by the concrete frames around the entire perimeter.

Concern: In order to perform properly, the masonry infill must contact the concrete framing elements on all four sides. Without proper attachment, the infill may not be able to provide the expected performance, and also may be subject to out-of-plane failure. This condition sometimes occurs when clerestory windows are provided at the top of the infilled panels.

Procedure: Recommend that positive connection between the infill and the frame be added.

Statement 7.3.5.18: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the capacity of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 7.3.5.19: No clay-tile arch floors are present.

Concern: Clay tile arch floor systems are heavy, brittle elements, whose seismic behavior is not well understood. Damage due to in-plane movements and vertical acceleration creates the potential for materials to fall from the slab underside. Solid brick arches are not of concern.

Procedure: Where clay tile arch floors exist, perform analyses for damage potential due to in-plane motion, using conservative values for allowable stresses. Evaluate the potential for damage to cause materials to fall from the slab underside.

Recommended C/D Ratio: 4.0.

7.3.5.3 Evaluation of Foundations

Statement 7.3.5.20: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 7.3.5.21: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 7.3.5.22: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 7.3.5.23: For buildings taller than six stories in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions (M > 5.5 high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

7.3.5.4 Evaluation of Non-Structural Elements

Statement 7.3.5.24: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 7.3.5.25: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.10 Tilt-Up Buildings with Precast Bearing Wall Panels (ATC-14 Section 8.1.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

8.1.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake trading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features, During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

Statement 8.1.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration. There is no substantial damage to wood elements due to bug infestation.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 8.1.5.2: There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil loop anchors have not been used.

Concern: Corrosion in post-tensioned anchorages can lead to the release of post-tensioning during ground shaking and cause failure of the lateral force resisting system. Coil loop anchors, which may be susceptible to failure, have been prohibited by current standards.

Procedure: Inspect a sample of the concrete in the area of the post-tensioning anchorage to determine its condition. Determine the extent and cause of the deterioration. Recommend that specific corrective action be taken.

Statement 8.1.5.3: There is no substantial damage to wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical loads carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Statement 8.1.5.4: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 8.1.5.5: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

8.1.5.2 Evaluation of Structural Elements

Statement 8.1.5.6: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 8.1.5.7: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 8.1.5.8: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement: 8.1.5.9: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

Concern: Heavy exterior walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during seismic response. If these walls are bearing walls, partial floor collapse may result. The hazard created increases with the height above the building base. The amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, and the type of soil.

Procedure: Calculate the inertial weight tributary to the anchorage level as the mass within one-half the distance between adjacent levels of anchorage. Use Equation 4.12 and table 4.8 to estimate the lateral force on this anchorage. If "government" anchors" are used for the wall anchorage, a testing program to determine this capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 4.0 for buildings with wood diaphragms (ABK, 1984); otherwise, 1.0.

Statement 8.1.5.10: The connection between the wall panels and the diaphragm does not induce cross grain bending or tension in the wood ledgers.

Concern: Cross grain tension can lead to abrupt, brittle failures in wood ledgers under actual yield level loads. These conditions are no longer permitted by the code.

Procedure: Recommend that connections be added that eliminate the cross grain bending condition.

Statement 8.1.5.11: The reinforcing steel for concrete walls is greater than .0025 times the gross area of the wall along both the longitudinal and transverse axes, at a spacing that does not exceed 18 inches.

Concern: A minimum amount of steel reinforcing is required for concrete walls to provide acceptable inelastic performance.

Procedure: Calculate the capacity of the walls with the reinforcing that is provided. Compute Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: $0.2\ R_w$.

Statement 8.1.5.12: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 8.1.5.13: All metal deck floors and roofs have a reinforced concrete topping slab with a minimum thickness of 3 inches.

Concern: Metal deck diaphragms without topping slabs may not have sufficient strength.

Procedure: Use an equivalent lateral force procedure to calculate a Capacity/Demand ratio for the strength of the bare metal deck diaphragm elements. The demand from this analysis should be compared with the minimum requirements for diaphragms given in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 8.1.5.14: Precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab with a minimum thickness of 3 inches that is doweled into the shear wall or frame elements.

Concern: Precast diaphragms without topping slabs may be susceptible to damage unless specifically detailed with connections capable of yielding or developing the strength of the connected elements.

Procedure: Use the equivalent lateral force procedure to calculate Capacity/Demand ratios of slab element interconnection. Check this force with the F_p force given in Equation 4.12.

Recommended C/D Ratio: 0.4 R_w.

Statement 8.1.5.15: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the strength of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 8.1.5.16: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include re-entrant corners may cause the wings of the structure to attempt to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

8.1.5.3 Evaluation of Foundations

Statement 8.1.5.17: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing.

Concern: Shear transfer for lateral loads between the wall panels and the foundation requires a continuous connection. Absence of such a connection can lead to panel rocking or sliding.

Procedure: Evaluate the Capacity/Demand ratio of the connection between the wall panels and the foundation.

Recommended C/D Ratio: 1.0.

Statement 8.1.5.18: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 8.1.5.19: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 8.1.5.20: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 8.1.5.21: For buildings in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions ($M > 5.5$ high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

8.1.5.4 Evaluation of Non-Structural Elements

Statement 8.1.5.22: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 8.1.5.23: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.11 Precast Concrete Frame and Concrete Shear Wall Buildings (ATC-14 Section 8.2.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

8.2.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$.10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause variations in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

8.2.5.1 Evaluation of Materials

Statement 8.2.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 8.2.5.2: There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil loop anchors have not been used.

Concern: Corrosion in post-tensioned anchorages can lead to the release of post-tensioning during ground shaking and cause failure of the lateral force resisting system. Coil loop anchors, which may be susceptible to failure, have been prohibited by current standards.

Procedure: Inspect a sample of the concrete in the area of the post--tensioning anchorage to determine its condition. Determine the extent and cause of the deterioration. Recommend that specific corrective action be taken.

Statement 8.2.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

8.2.5.2 Evaluation of Structural Elements

Statement 8.2.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures which have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 8.2.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effects of P-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 8.2.5.8: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

Concern: Heavy exterior walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during seismic response. If these walls are bearing walls, partial floor collapse may result. The hazard created increases with the height above the building base. The amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, and the type of soil.

Procedure: Calculate the inertial weight tributary to the anchorage level as the mass within one-half the distance between adjacent levels of anchorage. Use Equation 4.12 and Table 4.8 to estimate the lateral force on this anchorage. If "government anchors" are used for the wall anchorage, a testing program to determine this capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 4.0 for buildings with wood diaphragms (ABK, 1984); otherwise, 1.0.

Statement 8.2.5.9: The reinforcing steel for concrete walls is greater than.0025 times the gross area of the wall along both the longitudinal and transverse axes, at a spacing that does not exceed 18 inches.

Concern: A minimum amount of steel reinforcing is required for concrete walls to provide acceptable inelastic performance.

Procedure: Calculate the capacity of the walls with the reinforcing provided. Compute Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: 0.2 R_w.

Statement 8.2.5.10: The lateral force resisting elements form a well-distributed and balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities may cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 8.2.5.11: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e. a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic activity at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.12: All walls are continuous to the foundation.

Concern: Discontinuous walls can lead to column shear or axial load failures at the base of the discontinuous wall. Column failures can lead to full or partial collapse.

Procedure: Compare the column shear, moment, and axial force capacity at the discontinuity to the demands generated by the equivalent lateral force procedure. Check the diaphragm capacity to transfer these loads to other elements. Check the story stiffness to be sure that no soft story condition exists.

Recommended C/D Ratio: 0.4 R_w.

Statement 8.2.5.13: All metal deck floors and roofs have a reinforced concrete topping slab with a minimum thickness of 3 inches.

Concern: Metal deck diaphragms without topping slabs may not have sufficient strength.

Procedure: Use an equivalent lateral force procedure to calculate a Capacity/Demand ratio for the strength of the bare metal deck diaphragm elements. The demand from the analysis should be compared with the minimum requirements for diaphragms given in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.14: Precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab with a minimum thickness of 3 inches that is doweled and anchored into the shear wall or frame elements.

Concern: Precast diaphragms without topping slabs may be susceptible to damage unless specifically detailed with connections capable of yielding or developing the strength of the connected elements.

Procedure: Use the equivalent lateral force procedure to calculate Capacity/Demand ratios of diaphragm element interconnection. Check this force with the Fp force given in Equation 4.12.

Recommended C/D Ratio: 3.0.

Statement 8.2.5.15: If the frame girders bear on column corbels, the length of bearing is greater than 3 inches.

Concern: The maximum expected drift can be large, depending on the number and strength of the shear walls, the foundation conditions, and the relative rigidity of the diaphragms. Without specific calculation, interstory drifts of up to 3 inches should be accommodated. In precast buildings, if the girder shear connections fail, the corbel bearing area may need to be large enough to resist large local displacements.

Procedure: Use the equivalent lateral force procedure to estimate the interstory drift. Judge the adequacy of the precast connections to retain their vertical load carrying integrity at a maximum drift estimated to be 0.4 Rw times the calculating story drift.

8.2.6.5 Evaluation of Structural Details

Statement 8.2.5.16: There is reinforcing around all diaphragm openings that are larger than 50 percent of the building width in either major plan direction.

Concern: These large openings limit the strength of the diaphragm to transfer lateral forces.

Procedure: Verify the adequacy of the diaphragm to transfer stresses around the opening. Use the equivalent lateral force procedure and the provisions for diaphragms presented in Section 4.4.5.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.17: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include re-entrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.18: The diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length.

Concern: Shear wall buildings are dependent on diaphragms for proper performance. Substantial openings next to walls can prevent the proper transfer of load between the walls and the diaphragm.

Procedure: Verify that there is sufficient strength to deliver the appropriate amount of lateral load to the shear wall, using the equivalent lateral force procedure to calculate Capacity/Demand ratios for the load transfer.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.19: There is special wall reinforcement placed around all openings.

Concern: If the openings are not properly reinforced, they can reduce the strength of the walls. This can lead to degradation of the wall around the openings.

Procedure: Determine the capacity of the spandrels and piers considering all available reinforcing steel that crosses the critical sections. Calculate and evaluate Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

8.2.5.3 Evaluation of Foundations

Statement 8.2.5.20: All vertical wall reinforcing is doweled into the foundation.

Concern: The lack of sufficient dowels creates a weak plane that may not have adequate shear or tension capacity.

Procedure: Determine the dowel requirements from the ACI 318 minimum value (ACI, 1983) or the actual values from an analysis using the equivalent lateral force procedure and calculate Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.21: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 8.2.5.22: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 8.2.5.23: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 8.2.5.24: For buildings in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions (M > 5.5 high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

8.2.5.4 Evaluation of Non-Structural Elements

Statement 8.2.5.25: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 8.2.5.26: All exterior cladding, veneer courses, and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.12 Reinforced Masonry Buildings with Diaphragms of Wood or Metal Deck With or Without Concrete Fill (ATC-14 Section 9.1.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

9.1.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$ .10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause substantial variation in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

9.1.5.1 Evaluation of Materials

Statement 9.1.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration. There is no substantial damage to wood elements due to bug infestation.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 9.1.5.2: The mortar cannot be scraped away from the joints by hand with a metal tool, and there are no signs of eroded mortar.

Concern: Weak or eroded mortar indicates poor quality and possibly low strength for the walls.

Procedure: Estimate the compressive strength (f'm) of the masonry through testing. Determine the appropriate wall capacities from the test results and calculate the Capacity/Demand ratios.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.3: There is no substantial damage to wood or metal roof deck or structure due to roof leakage.

Concern: Persistent roof leakage can lead to substantial deterioration of roof decks and supporting members due to rotting of wood members, erosion of gypsum decks, and corrosion of steel decks and members. Both vertical loads carrying capacity and diaphragm capacity may be impaired.

Procedure: View roof decks for deterioration above ceiling spaces. Look particularly in areas where water stains are visible from below. Check particularly for highly stressed regions of the diaphragm such as at roof/wall connections. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Statement 9.1.5.4: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.5: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

9.1.5.2 Evaluation of Structural Elements

Statement 9.1.5.6: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.7: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 9.1.5.8: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effective of $P-\Delta$ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 9.1.5.9: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

Concern: Heavy exterior walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during seismic response. If these walls are bearing walls, partial floor collapse may result. The hazard created increases with the height above the building base. The amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, and the type of soil.

Procedure: Calculate the inertia weight tributary to the anchorage level as the mass within one-half the distance between adjacent levels of anchorage. Use Equation 4.12 and Table 4.8 to estimate the lateral force on this anchorage. If "government anchors" are used for the wall anchorage, a testing program to determine this capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 4.0 for buildings with wood diaphragms (ABK, 1984); otherwise, 1.0.

Statement 9.1.5.10: For buildings with wood diaphragms, the anchorage of exterior masonry walls is not accomplished by wood ledgers, which are subject to cross grain bending or cross grain tension.

Concern: Cross grain bending or tension can lead to abrupt, brittle failures in wood ledgers, which may be followed by wall or roof collapse.

Procedure: Recommend that anchorage be added that eliminates the cross grain bending condition.

Statement 9.1.5.11: The total vertical and horizontal reinforcing steel ratio is greater than .002 times the gross area of the wall, with a minimum of .0007 in either of the two directions. The spacing of reinforcing steel is less than 48 inches. All vertical bars extend to the top of the walls.

Concern: A minimum amount of steel and related grouted cells is required to provide the necessary performance.

Procedure: Calculate Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.12: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 R_w times maximum calculated drift for evaluation.

Statement 9.1.5.13: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e., a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic response at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.14: The anchors from the floor system into the exterior masonry walls are spaced at 4 feet or less.

Concern: The lack of sufficient wall anchors can cause partial collapse of the walls and adjacent floors due to out-of-plane forces.

Procedure: Calculate the Capacity/Demand ratios for the existing wall anchors using an equivalent lateral force procedure and the wall anchorage force, F_p, given by Equation 4.12.

Recommended C/D Ratio: 4.0.

Statement 9.1.5.15: The diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length.

Concern: Shear wall buildings are dependent on diaphragms for proper performance. Substantial openings next to walls can prevent the proper transfer of load between the walls and the diaphragms.

Procedure: Verify that there is sufficient strength to deliver the appropriate amount of lateral load to the shear wall using the equivalent lateral force procedure to calculate Capacity/Demand ratios for the load transfer.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.16: All wall openings that interrupt rebar have trim reinforcing on all sides.

Concern: To maintain the integrity of a nominally reinforced masonry wall with openings, trim rebar is required by the code and needed to resist diagonal cracking at corners and subsequent local deterioration.

Procedure: Use only the length of piers between reinforcing steel to calculate Capacity/Demand ratios from the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

9.1.5.3 Evaluation of Foundations

Statement 9.1.5.17: All vertical wall reinforcing is doweled into the foundation.

Concern: The lack of sufficient dowels creates a weak plane that may not have adequate shear or tension capacity.

Procedure: Determine the dowel requirements as the maximum of the ACl 318 minimum value (ACl, 1983) or the actual values from an analysis using the equivalent lateral force procedure and calculate Capacity/Demand ratios. The dowel capacity can be estimated by using shear friction concepts with a friction coefficient of 1.0.

Recommended C/D Ratio: 1.0.

Statement 9.1.5.18: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 9.1.5.19: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w

Statement 9.1.5.20: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Redommended C/D Ratio: 1.0.

Statement 9.1.5.21: For buildings taller than six stories in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions ($M > 5.5$ high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

9.1.5.4 Evaluation of Non-Structural Elements

Statement 9.1.5.22: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 9.1.5.23: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

6.13 Reinforced Masonry Bearing Wall Precast Concrete Diaphragm Buildings (ATC-14 Section 9.2.5)

The following pages should be used to replace the ATC-14 evaluation procedure for this building type in regions of low seismicity.

9.2.5 Evaluation of Buildings in Regions of Low Seismicity

The evaluation of any building is a complex task requiring the expertise of a professional engineer familiar with the seismic behavior of buildings. The procedure outlined below is intended to assist such an evaluation by flagging areas known to be potentially critical elements. Each is presented in terms of a statement, related concern and detailed evaluation procedure to be followed if the statement is not true for the building under study.

Each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study as noted. A false response is not to be interpreted as a condemnation of a building or element.

The final decision regarding the adequacy of a building is the responsibility of the reviewing structural engineer. This procedure is to be treated as a guide to that decision-making process and not as an absolute and/or short-cut method of evaluation.

The seismic evaluation of buildings in regions with EPA $\leq$ .10 g involves procedures similar to those required in regions of higher seismicity. The differences in seismicity cause substantial variation in the degree of complexity necessary to properly perform the different portions of the procedure.

In this hazard region, the evaluation of adequacy for earthquake loading need consider only the basic features of seismic resistance, such as the presence of a continuous load path for lateral forces, anchorage of parapets and other ornamentation, anchorage of exterior wall, cladding and veneer elements, and the basic competency of all materials that comprise elements of the vertical and lateral systems. For buildings in regions of low seismicity, the existence of these elements should result in a sufficiently low level of life-safety hazard.

The following statements address the basic items that should be included in the evaluation of this type of building in regions of low seismicity. It should be noted that these statements cover the items that may pose seismic hazards for buildings that do not possess any unusual features. During the data collection process (site visits and/or examination of construction documents), the engineer must be on the alert for any unusual building features that would pose additional seismic hazards. Familiarity with both the building performance characteristics and the items of concern for regions of higher seismicity is recommended to assist the evaluating engineer's judgment in identifying other potential seismic hazards.

9.2.5.1 Evaluation of Materials

Statement 9.2.5.1: The materials used to form the elements of both the vertical and lateral force resisting systems do not show signs of significant deterioration.

Concern: Deterioration of the structural materials may jeopardize the capacity of the vertical and lateral load resisting systems. This problem may become more prevalent for buildings located in severe climates where freeze/thaw cycles can lead to more rapid deterioration. All structural evaluations should include a site visit to determine the condition of the building.

Procedure: Identify all locations of significant deterioration to any of the structural elements participating in the lateral force resisting system, and recommend that corrective action be taken. If analyses of the existing conditions are performed, where deterioration is local, reduce or neglect the capacity of the deteriorated area. If the deterioration is extensive, materials testing should be performed.

Statement 9.2.5.2: The mortar cannot be scraped away from the joints by hand with a metal tool, and there are no signs of eroded mortar.

Concern: Weak or eroded mortar indicates poor quality and possibly low strength for the walls.

Procedure: Estimate the compressive strength (f'm) of the masonry through testing. Determine the appropriate wall capacities from the test results and calculate the Capacity/Demand ratios.

Recommended C/D Ratio: 1.0

Statement 9.2.5.3: Masonry and/or concrete elements have not been damaged by freeze/thaw action.

Concern: Cyclic freeze/thaw damage can substantially weaken masonry and concrete.

Procedure: Check all structural elements, particularly masonry and concrete walls, for spalling, crumbling, and scaling. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 9.2.5.4: Exposed concrete surfaces have not been damaged by chloride-laden concrete.

Concern: The presence of chloride in exposed concrete is widely known to cause severe damage to concrete and steel reinforcing. Parking garages are particularly susceptible to this phenomenon. The presence of chloride may be due to its addition to the concrete mix during construction or from the placement of deicing salts.

Procedure: Check exposed concrete surfaces for spalling, debonding, or corroded reinforcement. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0

9.2.5.2 Evaluation of Structural Elements

Statement 9.2.5.5: There is a complete lateral force resisting system that forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of the building together.

Concern: One of the fundamental attributes required for the proper response of a building during earthquake motions is that it is tied together to act as a single unit. The provision of a lateral system that ties all parts of the structure together and forms a complete system for resisting lateral loads is therefore necessary, even in regions of lowest seismicity. Often the strength of the elements provided to resist wind forces will be sufficient to withstand the loads produced by the design earthquake with an EPA of .10 g or less. However, the connections among all elements that comprise the load path may not be provided with sufficient capacity when subjected to seismic loads.

Procedure: For each major plan direction of the building, determine the load path for lateral forces. Check that there is a continuous load path from all diaphragm levels to the vertical shear resisting elements (such as frames or walls) to the foundation and into the surrounding soil. The capacity of the connections between the major elements in the lateral force resisting system should be checked for a lateral load of 5 percent of the dead and live load tributary to the area resisted by the elements under consideration.

Recommended C/D Ratio: 1.0.

Statement 9.2.5.6: The building has been provided with a redundant system such that the failure of a single member, connection, or component does not adversely affect the lateral stability of the structure.

Concern: In structures that have not been provided with redundancy, all components must remain operative for the structure to retain its lateral stability. Because of the uncertainties involved in the magnitude of both the seismic loads and the member capacities, the provision of redundancy is recommended.

Procedure: Check that the stability of the lateral force resisting system does not rely on any single component or connection. If the building is not redundant, recommend that additional lateral force resisting elements be added.

Statement 9.2.5.7: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Soft stories (i.e. a decrease in story yield capacity of more than 20 percent from one story to the story immediately below) or other severe vertical strength irregularities can cause a concentration of inelastic response, interstory drift, and nonstructural damage.

Procedure: Use the equivalent lateral force procedure to determine the distribution of lateral forces and consider the additive effective of $P-\Delta$ stresses.

Recommended C/D Ratio: $0.4\ R_w$.

Statement 9.2.5.8: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

Concern: Heavy exterior walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during seismic response. If these walls are bearing walls, partial floor collapse may result. The hazard created increases with the height above the building base. The amplification of the ground motion used to estimate the wall anchorage forces depends on the type and configuration of both the walls and the diaphragms, and the type of soil.

Procedure: Calculate the inertia weight tributary to the anchorage level as the mass within one-half the distance between adjacent levels of anchorage. Use Equation 4.12 and Table 4.8 to estimate the lateral force on this anchorage. If "government anchors" are used for the wall anchorage, a testing program to determine this capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 4.0 for buildings with wood diaphragms (ABK, 1984); otherwise, 1.0.

Statement 9.2.5.9: The total vertical and horizontal reinforcing steel ratio is greater than .002 times the gross area of the wall, with a minimum of .0007 in either of the two directions. The spacing of reinforcing steel is less than 48 inches. All vertical bars extend to the top of the walls.

Concern: A minimum amount of steel and related grouted cells is required to provide the necessary performance.

Procedure: Calculate Capacity/Demand ratios using the equivalent lateral force procedure.

Recommended C/D Ratio: 1.0.

Statement 9.2.5.10: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan direction.

Concern: Plan irregularities can cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Procedure: Verify the adequacy of the system by analyzing the torsional response using procedures that are appropriate for the relative rigidities of the diaphragms and the vertical elements. Compare the maximum calculated story drift with 0.00511. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 1.4 Rw times maximum calculated drift for evaluation.

Statement 9.2.5.11: There are no significant vertical irregularities caused by either geometric or mass irregularities.

Concern: Vertical irregularities caused by setbacks (i.e., a change in a horizontal dimension of the lateral force resisting system of more than 30 percent in a story relative to the adjacent stories) or mass irregularities (i.e., a change in the effective mass of more than 50 percent from one story to the next, excluding lighter roofs) produces distribution of the base shear that can be significantly different from that of regular buildings. This can lead to a concentration of inelastic activity at the location of the irregularity.

Procedure: Use the modal analysis procedure given in Section 4.4.4 to determine a more realistic distribution of the base shear. Calculate Capacity/Demand ratios for the lateral force resisting elements.

Recommended C/D Ratio: 1.0.

Statement 9.2.5.12: The topping slab with a minimum thickness of 3 inches continues uninterrupted through the interior walls and into the exterior walls or is provided with dowels of a total area equal to the topping slab reinforcing.

Concern: The topping slab may not be fully effective if it is interrupted at interior walls. When topping slab steel is not continuous through the interior walls, the diaphragm strength and ductility may be severely limited. Tension failure at an interior wall could result in floor spreading and possibly partial collapse. Exterior walls may collapse if not well anchored to the wall.

Procedure: Evaluate the tension and shear demand due to diaphragm forces, including collector requirements, perpendicular to wall loads, or chord forces at re-entrant corners. Determine the Capacity/Demand ratios using the equivalent lateral force procedure and the diaphragm requirements given by Equation 4.13.

Recommended C/D Ratio: 0.2 R_w.

Statement 9.2.5.13: The anchors from the floor system into the exterior masonry walls are spaced at 4 feet or less.

Concern: The lack of sufficient wall anchors can cause partial collapse of the walls and adjacent floors due to out-of-plane forces.

Procedure: Calculate the Capacity/Demand ratios for the existing wall anchors using an equivalent lateral force procedure and the wall anchorage force, F_p, given by Equation 4.12.

Recommended C/D Ratio: 4.0.

Statement 9.2.5.14: The diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length.

Concern: Shear wall buildings are dependent on diaphragms for proper performance. Substantial openings next to walls can prevent the proper transfer of load between the walls and the diaphragms.

Procedure: Verify that there is sufficient strength to deliver the appropriate amount of lateral load to the shear wall using the equivalent lateral force procedure to calculate Capacity/Demand ratios for the load transfer.

Recommended C/D Ratio: 1.0.

Statement 9.2.5.15: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities.

Concern: Buildings with substantial plan irregularities that include re-entrant corners may cause the wings of the structure to vibrate independently. If the tensile capacity provided at the re-entrant corners is not sufficient to restrict this motion, a local concentration of damage, including partial collapse, may occur.

Procedure: Evaluate the chord/collector requirements at the re-entrant corners by applying the maximum of the diaphragm force suggested in Section 4.4.5 and the calculated story acceleration to a model of the isolated floor diaphragm. All elements that can contribute to the tensile capacity at the re-entrant corner may be included with appropriate consideration given to gravity load stresses.

Recommended C/D Ratio: 1.0.

9.2.5.3 Evaluation of Foundations

Statement 9.2.5.16: All vertical wall reinforcing is doweled into the foundation.

Concern: The lack of sufficient dowels creates a weak plane that may not have adequate shear or tension capacity.

Procedure: Determine the dowel requirements as the maximum of the ACl 318 minimum value (ACl, 1983) or the actual values from an analysis using the equivalent lateral force procedure and calculate Capacity/Demand ratios. The dowel capacity can be estimated by using shear friction concepts with a friction coefficient of 1.0

Recommended C/D Ratio: 1.0.

Statement 9.2.5.17: If the pile foundation of the building extends above grade, such as in coastal regions or plains, the lateral stiffness and strength of the foundation is no less than that of the structure above the foundation.

Concern: A typical construction practice for regions subjected to flooding is to extend a pile foundation above the high water level. If the foundation system is not of sufficient strength or stiffness, a soft story condition results which can cause a concentration of inelastic activity, and interstory drift, and even collapse.

Procedure: Evaluate the building above the foundation according to the appropriate model buildings(s) evaluation procedure. Distribute the base shear and overturning forces for the building to evaluate the capacity of the foundation structure. The foundation structure should be modeled as a moment frame or braced frame system, depending on the configuration of the piling. The soil-pile interaction should be considered in determining the base fixity.

Recommended C/D Ratio: 0.2 R_w for wood pile systems; 0.4 R_w otherwise.

Statement 9.2.5.18: The foundation of the building is not composed of unreinforced masonry or stone rubble.

Concern: Unreinforced masonry and stone rubble foundations may not have sufficient capacity to properly transfer the lateral forces between the soil and the structure above. These areas may also be subjected to more rapid deterioration because they are located below grade.

Procedure: Evaluate the capacity of the foundation to transmit the lateral forces, considering the present state of deterioration of the foundation elements.

Recommended C/D Ratio: 0.4 R_w.

Statement 9.2.5.19: There is no foundation or superstructure damage due to heaving soil.

Concern: Soil heaving due to freezing can substantially damage foundations and superstructures.

Procedure: Check for heaving damage in foundations and superstructures. This damage usually manifests itself in the form of step cracking. In capacity calculations, reduce the capacity at areas of local deterioration. If the deterioration is extensive, materials testing should be performed.

Recommended C/D Ratio: 1.0.

Statement 9.2.5.20: For buildings in regions which can generate an earthquake larger than a magnitude of 5.5 (EPA $\geq$ .10 g), the structure is not founded on fine grain sands where the groundwater table is less than 30 feet from the surface.

Concern: Buildings founded on fine grained soils with high groundwater tables may be subject to liquefaction during earthquakes of magnitude larger than 5.5. This type of building may be subject to severe damage if significant differential settlements occur as a result of liquefied soil. Note that all three conditions (M > 5.5 high groundwater table and fine grained soils) must be present to create a situation where liquefaction is possible.

Procedure: Review the information of liquefaction in Section 3.2 of this report. Perform a preliminary evaluation of the liquefaction potential using the procedure presented in Appendix B. If this analysis indicates a potential for liquefaction, recommend that the owner retain the services of a qualified geotechnical engineer to perform an in-depth study of the liquefaction potential.

9.2.5.4 Evaluation of Non-Structural Elements

Statement 9.2.5.21: All cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces are reinforced and anchored to the structure.

Concern: Cornices, parapets, and other ornamentation that are not reinforced and anchored to the building can create significant falling hazards. The hazard created increases with the height above the building base.

Procedure: Determine the anchorage capacity and use Equation 4.12 and Table 4.8 to estimate the appropriate anchorage force. If "government anchors" are used, a testing program to determine their capacity is recommended. Figure 6.1 depicts a typical detail for a "government anchor".

Recommended C/D Ratio: 1.0.

Statement 9.2.5.22: All exterior cladding, veneer courses and/or exterior wall courses above the first story or above 12'-0" are properly anchored to the exterior wall framing.

Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

Procedure: Estimate the anchorage force required for cladding and veneer by using Equation 4.12 and Table 4.8. If corrugated metal ties are used for anchorage, a testing program to determine their capacity is recommended.

Recommended C/D Ratio: 1.0.

CHAPTER 7

REVISIONS AND ADDITIONS TO THE ATC-14 CHAPTER ON THE SEISMIC EVALUATION OF NON-STRUCTURAL ELEMENTS

Chapter 11 of ATC-14 addresses the seismic evaluation of non-structural elements. It is based on the General Services Administration's guidelines for the seismic evaluation of existing buildings. A list of performance characteristics for typical non-structural elements is provided as the basis for a set of non-structural checklist statements. The statements in this section are not provided with "Concerns" and "Procedures" similar to those which accompany the structural evaluation statements. Many of these issues identify potential damage which would most likely not constitute a threat to life safety. Others which could pose such a threat are identified to alert the evaluator.

The panel members also made a number of suggestions for revisions and additions to this Chapter of ATC-14. More detailed descriptions and the addition of a number of figures were suggested in order to clarify some of the issues presented in this Chapter. In addition, the panel members recommended major expansions of the sections on elevators and exterior cladding. A new section to address building contents was also recommended.

As a result of these recommendations, a significant modification of Chapter 11 was performed. Because of the large scope of these suggested modifications, the amended Chapter 11 has been included in its entirety on the following pages.

CHAPTER 11

SEISMIC EVALUATION OF NON-STRUCTURAL ELEMENTS

When damage to non-structural elements is of concern to the building owner, the evaluating engineer will need to include the evaluation of non-structural elements as part of the overall building evaluation. The sources of information for evaluating non-structural elements are similar to those used in the structural evaluation (See Section 4.2). The non-structural evaluation methodology includes consideration of performance characteristics as well as a review of a list of evaluation statements similar to those presented for each of the model building types (Chapter 5 through 10). Of particular importance in the non-structural element evaluation efforts are site visits to identify the present status of non-structural items; this effort will take on added importance because non-structural elements of structures may be modified many times during the life of the structure.

Performance characteristics applicable for severe earthquake shaking are listed in the following section for all major types of non-structural elements (e.g., partitions, ceilings, etc.). This list is based on Volume III of the General Services Administration's procedure for evaluating existing buildings (GSA, 1976). It is not meant to be exhaustive, but rather representative of the type of performance that can be expected. It should be noted that non-structural elements can pose significant hazards to life safety under certain circumstances. All performance characteristics which could pose such a threat to life safety are designated with the symbol (LS). Special or customized building contents that could present hazards, such as toxic chemicals, should also be considered in the evaluation of non-structural components. Special consideration may be necessary for non-structural elements in essential facilities such as hospitals, police and fire stations, and other facilities which should remain in operation after an earthquake. Three other references (McGavin, 1981, Reitherman, 1980 and Veterans Administration, 1976) also provide a great deal of information on this subject.

Following the performance characteristics are lists of evaluation statements. As in the case for each model building type, each statement is structured such that a "true" response implies no further study is required. A "false" response implies that the designated element needs further study, but is not to be interpreted as a condemnation of the element. In addition to this list, the building evaluation procedures (Chapters 5 through 10) usually include non-structural considerations that should be addressed in all structural evaluations.

11.1 Performance Characteristics of Typical Non-Structural Elements

This section presents a list of typical non-structural elements and the performance characteristics that each are expected to exhibit during seismic events.

11.1.1 Partitions

1. Masonry and Tile. These partitions can have severe cracking or loss of units. Compression failures can occur at the tops of the partitions, or at the joints. These partitions may collapse and fail due to perpendicular-to-wall loads. (LS)

2. Stud and Gypsum Board or Plaster. These partitions may overturn due to local ceiling failures. Finishes may crack or detach from the studs.

3. Demountable Partitions of Metal, Wood, and/or Glass. These partitions may separate from the supporting channels possibly resulting in overturning. Fixed glass may crack or separate from remainder of partition.

11.1.2 Furring

The plaster or gypsum board finishes may crack or separate from the furred structural elements.

11.1.3 Ceilings

1. Suspended Lay-in Tile Systems. Hangers may unwind or break. Tiles may separate from suspension system and fall. Breakage may also occur at seismic joints and at building perimeters.

2. Suspended Plaster or Gypsum Board. Plaster may have finish cracks that could lead to spalling. Hangers may break. Gypsum board or plaster may separate from suspension system and fall.

3. Surface Applied Tile, Plaster, or Gypsum Board. Plaster may crack and spall. Ceiling tiles may fall due to adhesive failures.

11.1.4 Light Fixtures

1. Lay-in Fluorescent. Ceiling movement can cause fixtures to separate and fall from suspension systems. Parts within the fixtures are prone to separate from the housing. These systems perform better when they are supported separately from the ceiling system, or have back-up support that is independent of the ceiling system.

2. Stem or Chain Hung Fluorescent. The stem connection to structural elements may fail. Fixtures may twist severely, causing breakage in stems or chains. Long rows of fixtures placed end to end are often damaged due to the interaction. Long stem fixtures tend to suffer more damage than short stem units. Parts within the fixture may separate from the housing and fall.

3. Surface Mounted Fluorescent. Ceiling mounted fixtures perform in a fashion similar to lay-in fixtures. Wall fixtures generally perform better than ceiling fixtures. Parts within the fixture may separate from the housing and fall.

4. Stem Hung Incandescent. These fixtures are usually suspended from a single stem or chain that allows them to sway. This swaying may cause the light and/or the fixture the break after encountering other structural or non-structural components.

5. Surface Mounted Incandescent. Ceiling movement can cause fixtures to separate and fall from suspension systems. Wall mounted fixtures performed well.

11.1.5 Doors and Frames

Frames can warp from wall deformations, possibly causing the door to bind.

11.1.6 Mechanical Equipment

1. Rigidly Mounted Large Equipment (Boilers, chillers, tanks, generators, etc.). Shearing of anchor bolts can occur and lead to horizontal motion. Unanchored equipment will move and damage connecting utilities. Tall tanks may overturn. Performance is generally good when positive attachment to the structure is provided.

2. Vibration Isolated Equipment (Fans, pumps, etc.). Isolation devices can fail and cause equipment to fall. Unrestrained motion can lead to damage. Suspended equipment is more susceptible to damage than mounted equipment. (LS)

11.1.7 Piping

Large diameter rigid piping can fail at elbows, tees, and at connections to supported equipment. Joints may separate and hangers may fail. Hanger failures can cause progressive failure of other hangers or supports. Failures may occur in pipes that cross seismic joints due to differential movements and adjacent rigid supports. The increased flexibility of small diameter pipes often allows them to perform better than larger diameter pipes, although they are subject to damage at the joints. Piping in vertical runs typically performs better than in horizontal runs if regularly connected to a vertical shaft.

11.1.8 Ducts

Breakage is most common at bends. Supporting yokes may also fail at connection to the structural element. Failures may occur in long runs due to large amplitude swaying. Failure usually consists of leakage only and not collapse.

11.1.9 Electrical Equipment

Tall panels may overturn when they are not bolted or braced. Equipment may move horizontally if not positively anchored to the floor.

11.1.10 Elevators

1. Counterweights and Guiderails. Counterweights may separate from rails. Counterweights may also damage structural members, cables, and cabs. (LS)

2. Motor/Generator. The motor (or generator) may shear off the vibration isolators.

3. Control Panels. Control panels can overturn when they are not anchored.

4. Cars and Guiding Systems. Cars and guiding systems generally perform well, except that cables may separate from drums and sheaver.

5. Hoistway Doors. Doors can jam or topple due to shaking or excessive drift.

6. Hydraulic Elevator Systems. These systems usually perform well except that the cylinders may shift out-of-plumb.

11.1.11 Exterior Cladding/Glazing or Veneers

1. Exterior wall panels or cladding can fall onto the adjacent property if their connection to the building frames have insufficient strength and/or ductility. (LS)

2. If glazing is not sufficiently isolated from structural motion, or above 12'0", it can shatter and fall onto adjacent property.

11.1.12 Parapets, Cornices, Ornamentation and Appendages

1. If any of these items are of insufficient strength and/or are not securely attached to the structural elements, they may break off and fall onto storefronts, streets, sidewalks, or adjacent property. (LS)

11.1.13 Means of Egress

1. Hollow tile or unreinforced masonry walls often fail and litter stairs and corridors. (LS)

2. Stairs connected to each floor can be damaged due to interstory drift, especially in flexible structures such as moment frame buildings. (LS)

3. Veneers, cornices, ornaments, and canopies over exits can fall and block egress. (LS)

4. Corridor and/or stair doors may jam due to partition distortion. (LS)

5. Lay-in ceiling tiles and light fixtures can fall and block egress. (LS)

11.1.14 Building Contents and Furnishings

1. Desk-Top Equipment. Desk-top equipment such as typewriters, computers, etc., may slide off and fall if they are not sufficiently anchored to the desk.

2. File Cabinets. Tall file cabinets may tip over and fall if they are not anchored to resist overturning forces. Unlatched cabinet drawers may slide open and fall.

3. Storage Cabinets and Racks. Tall, narrow storage cabinets or racks can tip over and fall if they are not anchored to resist overturning forces. (LS)

4. Plants, Artwork and Other Objects. Plants, artwork and other objects which are located on top of desks, cabinets, etc., can fall if they are not anchored to resist their lateral movement.

5. Items Stored on Shelves. Items stored on shelving such as in laboratories or retail stores can fall if they are not restrained from sliding off the shelves.

6. Computers and Communications Equipment. Tall, narrow equipment can overturn and fall if they are not anchored to resist overturning forces. (LS)

7. Computer Access Floors. Unbraced computer floors can roll off their supports and fall to the structural slab.

11.1.15 Hazardous Materials

Because of the secondary dangers which can result from damage to vessels which contain hazardous materials, special precautions should be considered for the proper bracing and restraint of these elements.

1. Compressed Gas Cylinders. Unrestrained compressed gas cylinders can be damaged such that the gas is released and/or ignited. (LS)

2. Laboratory Chemicals. Unrestrained chemicals can mix and react if they are spilled. (LS)

3. Piping. Piping which contains hazardous materials can leak if shut-off valves or other devices are not provided. (LS)

11.2 Evaluation of Non-structural Elements

Included herein are evaluation statements for each of the non-structural items listed above. Each statement is designed to expose potential damage in regions of high or moderate seismicity. Any statement in the list that is designated with an (LS) is concerned with a possible life-safety issue. Other statements in the list are also concerned with damage, but are not considered to pose a life-safety hazard except in rare cases. When a building has features that could cause non-structural damage (i.e., the answer to the statement is "false"), the procedures suggested in Section 4.4.5 can be used to calculate Capacity/Demand ratios. The recommended Capacity/Demand ratios should be taken as 1.0 for items that are perceived to be ductile, and 0.4 Rw for elements thought to fail in a brittle manner. Calculation of these Capacity/Demand ratios is recommended for all elements given the (LS) designation. If possible life-safety hazards are identified, the engineer should inform the owner of this condition and recommend that corrective action be taken. For other types of non-structural damage, the owner should be informed.

11.2.1 Partitions

1. All unreinforced masonry or hollow clay tile are 8 feet tall or less. See Sections 6.5, 7.3, or Chapter 10 for evaluation of unreinforced masonry buildings. (LS)

2. The partitions are detailed to accommodate the expected interstory drift.

3. None of the partitions cross seismic joints.

4. For partitions that only extend to the ceiling line, there is lateral bracing for the top of the partitions. See Figure 11.1 for a reinforced masonry partition with lateral bracing at the ceiling level.

11.2.2 Furring

None of the structural elements are furred.

11.2.3 Ceilings

1. The ceilings are not suspended plaster or gypsum board. See Figure 11.2 for proper bracing details for suspended ceilings.

2. Clips are not used for attachment of ceiling panels or tiles.

3. Lay-in tiles are not used for ceiling panels.

4. The ceiling system does not extend continuously across any of the seismic joints.

5. The ceiling system is not required to laterally support the top of masonry, gypsum board, or hollow clay tile partitions.

6. The edges of ceilings are separated from structural walls.

11.2.4 Light Fixtures

1. Multiple length fluorescent fixtures have bracing or secondary support throughout their length. See Figure 11.3 for typical bracing details for these fixtures.

2. The lenses on fluorescent light fixtures are supplied with safety chains or some form of positive attachment.

3. Pendant fixtures are not close enough to come into contact with any structural or other non-structural elements.

4. Double stem fluorescent fixtures are not used. See Figure 11.3.

11.2.5 Mechanical Equipment

1. There is positive attachment of large equipment to the structural system, by means of anchor bolts or some other method. Tall, narrow panels (H/D > 3, e.g.) may require anchorage at the top in addition to the base attachment.

2. The vibration isolated pieces of equipment are provided with restraints to limit horizontal and vertical motion. See Figure 11.4 for a typical restraint detail.

3. None of the major mechanical equipment items are suspended from the ceiling without seismic bracing. See Figure 11.5 for a properly braced piece of suspended equipment.

11.2.6 Piping

1. None of the pipes cross seismic joints without a flexible connector.

2. No pipes are supported by other pipes.

3. None of the pipe sleeve wall openings have diameters less than about two inches larger than the pipe.

11.2.7 Ducts

1. Duct work in long lines is laterally braced along its entire length. See 11.6 for a properly braced duct line.

2. None of the ducts are supported by piping or other non-structural elements.

3. Ducts have flexible sections crossing seismic joints.

11.2.8 Electrical Equipment

1. All of the electrical equipment is positively attached to the structural system, by means of anchor bolts or some other method. Tall, narrow panels (H/D > 3, e.g.) may require anchorage at the top in addition to the base anchorage.

2. All equipment supported on access floor systems are either directly attached to the structure or are fastened to a laterally braced floor system. See Figure 11.7.

11.2.9 Elevators

1. All elements of the elevator support system are adequately anchored and configured to resist lateral seismic forces. These elements are as shown in Figure 11.8 and include the car and counterweight frames, guides, guide rails, supporting brackets and framing, driving machinery, operating devices, and control equipment. (LS)

2. With the elevator car and/or counterweight located in its most adverse position in relation to the guide rails and support brackets, the horizontal deflection will not exceed 1/2 inch between supports and horizontal deflections of the brackets will not exceed 1/4 inch. Use Formula (4.12) in computing the loads assuming $C_p = 0.30$ and that the lateral forces acting on the guide rails will be assumed to be distributed 1/3 to the top guide rollers and 2/3 to the bottom guide rollers of the elevator car and counterweights. (LS)

3. Cable retainer guards on sheaves and drums were installed as required to inhibit the displacement of cables.

4. Snag points created by rail brackets, fish plates, etc., are equipped with guards as required to prevent snagging of relevant moving elements. (LS)

5. The clearance between the car and counterweight assembly and between the counterweight assembly and the hoistway enclosure or separator beam is not less than 2 inches. (LS)

6. The maximum spacing of the counterweight rail tie brackets tied to the building structure does not exceed 16 feet. An intermediate spreader bracket is provided for tie brackets spaced greater than 10 feet and two intermediate spreader brackets are provided for tie brackets greater than 14 feet. (LS)

7. A retainer plate is provided at top and bottom of both car and counterweight. The clearance between the faces of the rail and the retainer plate does not exceed 3/16 inches.

8. The control panels are bolted to the floor slabs.

11.2.10 Cladding, Glazing and Veneer

1. Materials

(1) There is no substantial damage to the exterior cladding due to water leakage.

Concern: Water leakage into and through exterior walls is a common building problem. Damage due to corrosion, rotting, freezing, or erosion can be concealed within wall spaces. Substantial deterioration can lead to loss of cladding elements or panels.

Procedure: Check exterior walls for deterioration, probing into wall space if necessary. Look for signs of water leakage at vulnerable interior spaces, such as around windows and at floor areas. Particularly check ties of cladding elements to the backup structure and ties of the backup structure to floor and roof slabs.

(2) There is no damage to exterior wall cladding due to temperature movements.

Concern: Extremes of temperature can cause substantial structural damage to exterior walls. The resulting weakness may be brought out in a seismic event.

Procedure: Check exterior walls for cracking due to thermal movements.

2. <u>Brick Veneer with Concrete Block Backup</u>

(1) The brick veneer is supported by shelf angles or other element at each floor level. (LS)

(2) The brick veneer is adequately anchored to the backup at locations of through-wall flashing. (LS)

(3) Brick veneer is connected to the backup with ties at 24 inch o.c. maximum and with one tie every 2-2/3 foot square maximum. (LS)

(4) The concrete block backup qualifies as reinforced masonry (high seismicity only). (LS)

(5) The concrete block backup is positively anchored to the structural frame at 4'-0" maximum. (LS)

(6) For moment frame buildings of steel or concrete (Sections 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(7) All eccentricities in connections are accounted for. (LS)

(8) Connections appear to be installed generally in accordance with the construction documents. (LS)

(9) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(10) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(11) There is no cracking in the brick veneer indicative of substantial structural distress. (LS)

(12) Mortar joints in brick and block wythes are well-filled, and material cannot be easily scraped from the joints. (LS)

3. Brick Veneer with Steel Stud Backup

(1) The brick veneer is supported by shelf angles or other elements at each floor level. (LS)

(2) The brick veneer is adequately anchored to the backup in the vicinity of locations of through-wall flashing. (LS)

(3) Brick veneer is connected to the backup with ties at 24 inches o.c. maximum and with one tie every 2-2/3 foot square maximum. (LS)

(4) For moment frame buildings of steel or concrete (Sections 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(5) Corrugated brick ties are not used. (LS)

(6) All eccentricities in connections are accounted for. (LS)

(7) Connections appear to be installed generally in accordance with the construction documents.

(8) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(9) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(10) There is no cracking in the brick veneer indicative of substantial structural distress. (LS)

(11) Computed tensile stresses in the veneer do not exceed the allowable (as defined by the Brick Institute of America) using Cp = 0.75 and C/D = 4. (LS)

(12) Mortar joints in the brick veneer are well filled, and material cannot be easily scraped out from the joints. (LS)

(13) Additional steel studs frame window and door openings. (LS)

(14) There is no visible corrosion of brick ties, tie screws, studs, or stud tracks. (LS)

(15) There is no visible deterioration of exterior sheathing. (LS)

(16) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

4. Precast Concrete

(1) There are at least two bearing connections for each wall panel. (LS)

(2) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(3) Where multi-story panels are attached at each floor level, the panels and connections can accommodate an interstory drift of three inches without collapse. (LS)

(4) For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(5) Where inserts are used in concrete connections, the inserts are welded to or hooked around reinforcing steel. (LS)

(6) All eccentricities in connections are accounted for. (LS)

(7) Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts. (LS)

(8) Connections appear to be installed generally in accordance with the construction documents. (LS)

(9) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(10) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

5. Thin Stone Veneer Panels

(1) There are at least two bearing connections for each wall panel. (LS)

(2) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(3) For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, the panels and connections can accommodate an interstory drift of three inches without collapse. (LS)

(4) For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(5) Where inserts are used in concrete connections, the inserts are welded to or hooked around reinforcing steel. (LS)

(6) All eccentricities in connections are accounted for. (LS)

(7) Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts. (LS)

(8) Connections appear to be installed generally in accordance with the construction documents. (LS)

(9) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(10) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(11) Stone anchorages are adequate for computed loads using Cp = 0.75 and C/D = 4. (LS)

(12) There are no visible cracks or weak veins in the stone. (LS)

6. Glass and Metal Curtainwall Panels

(1) There are at least two bearing connections for each curtain wall panel. (LS)

(2) There are at least four connections for each curtain wall panel capable of resisting out-of-plane forces. (LS)

(3) For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, the panels and connections can accommodate an interstory drift of three inches without collapse. (LS)

(4) For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(5) All eccentricities in connections are accounted for. (LS)

(6) Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts. (LS)

(7) Connections appear to be installed generally in accordance with the construction documents. (LS)

(8) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(9) Where inserts are used in concrete connections, the inserts are welded to or hooked around reinforcing steel. (LS)

7. Wood/Aggregate Panels

(1) There are at least two bearing connections for each wall panel. (LS)

(2) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(3) For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, the panels and connections can accommodate an interstory drift of three inches without collapse. (LS)

(4) For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(5) All eccentricities in connections are accounted for. (LS)

(6) Connections appear to be installed generally in accordance with the construction documents. (LS)

(7) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(8) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(9) Additional steel studs frame window and door openings. (LS)

(10) There is no visible corrosion of tie screws, studs, or stud tracks. (LS)

(11) There is no visible deterioration of exterior sheathing. (LS)

(12) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

(13) There is no visible deterioration of screws or wood at panel attachment points. (LS)

8. Stucco Finish on Lath Panels

(1) For moment frame buildings of steel or concrete (Sections 6.1 and 7.1), where multi-story panels are attached at each floor level, the panels and connections can accommodate an interstory drift of three inches without collapse. (LS)

(2) For moment frame buildings of steel or concrete (Sections 6.1 and 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift to three inches without collapse. (LS)

(3) All eccentricities in connections are accounted for. (LS)

(4) Connections appear to be installed generally in accordance with the construction documents. (LS)

(5) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(6) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(7) Additional steel studs frame window and door openings. (LS)

(8) There is no visible corrosion of tie screws, studs, or stud tracks. (LS)

(9) There is no visible deterioration of exterior sheathing. (LS)

(10) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

9. <u>Composite Expanded Polystyrene and Stucco Panels</u>

(1) There are at least two bearing connections for each wall panel. (LS)

(2) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(3) For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, the panels and connections can accommodate an interstory drift of three inches without collapse. (LS)

(4) For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels are sufficiently isolated from the structural frame to absorb an interstory drift of three inches without collapse. (LS)

(5) Connections appear to be installed generally in accordance with the construction documents. (LS)

(6) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(7) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(8) Additional steel studs frame window and door openings. (LS)

(9) There is no visible corrosion of tie screws, studs, or stud tracks. (LS)

(10) There is no visible deterioration of exterior sheathing. (LS)

(11) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

11.2.11 Parapets, Cornices, Ornamentation, and Appendages

1. There are no laterally unsupported unreinforced masonry parapets or cornices above the highest level of anchorage with height/thickness ratios greater than 1.5. A typical parapet bracing detail is shown in Figure 11.9.

2. There are no laterally unsupported reinforced masonry parapets or cornices above the highest anchorage level with height/thickness ratios greater than 3. (LS)

3. Concrete parapets with height/thickness ratios greater than 1.5 have vertical reinforcement. (LS)

4. All appendages or other exterior wall ornamentations are well anchored to the structural system. (LS)

11.2.12 Means of Egress

1. The walls around stairs and corridors are of a material other than hollow clay tile or unreinforced masonry. (LS)

2. All veneers, parapets, cornices, canopies, and other ornamentation above building exits are well anchored to the structural system. (LS)

3. Lay-in ceiling tiles are not used in exits or corridors. (LS)

11.2.13 Building Contents and Furnishings

1. All desk-top equipment is anchored to restrain it from sliding off the desk.

2. All tall file cabinets are anchored to the floor slab or an adjacent partition wall. File cabinets arranged in groups are attached together to increase their stability. Cabinet drawers have latches to keep them closed during shaking.

3. Tall, narrow (H/D > 3) storage racks are anchored to the floor slab or adjacent walls. (LS)

4. Plants, artwork and other objects are anchored to restrict their motion.

5. All breakable items stored on shelves are restrained from falling by latched doors, shelf lips, wires, or other methods.

6. Computers and Communications equipment are anchored to the floor slab and/or structural walls to resist overturning forces. See Figure 11.7. (LS)

7. Computer access floors are braced to resist lateral forces. See Figure 11.7.

11.2.14 Hazardous Materials

1. Compressed gas cylinders are restrained against motion. (LS)

2. Laboratory chemicals stored breakable containers are restrained from falling by latched doors, shelf lips, wires or other methods. (LS)

3. Piping containing hazardous materials is provided with shut-off valves or other devices to prevent major spills or leaks. (LS)

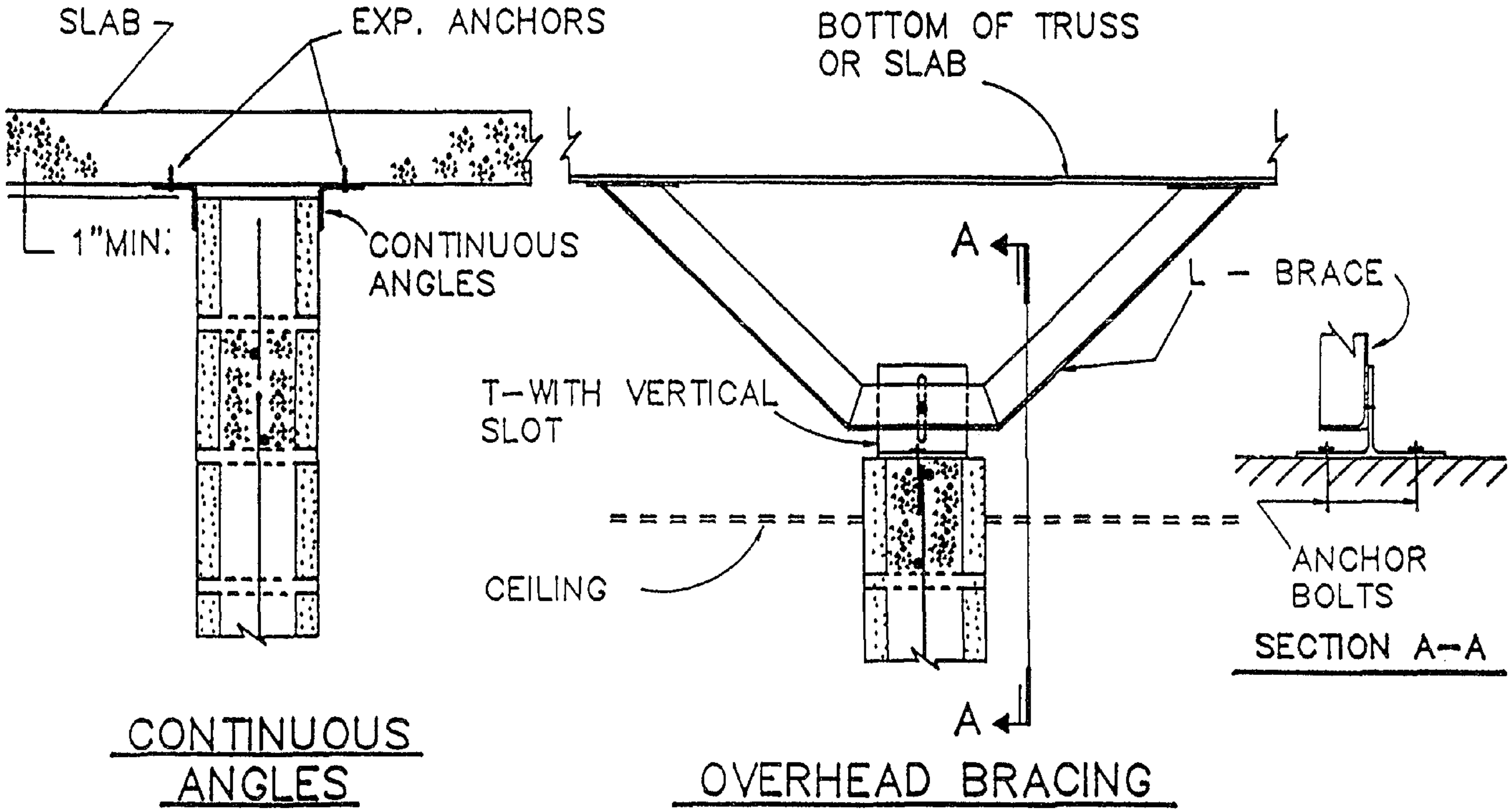


Figure 11.1 - Typical Details for Bracing Masonry Partition Walls

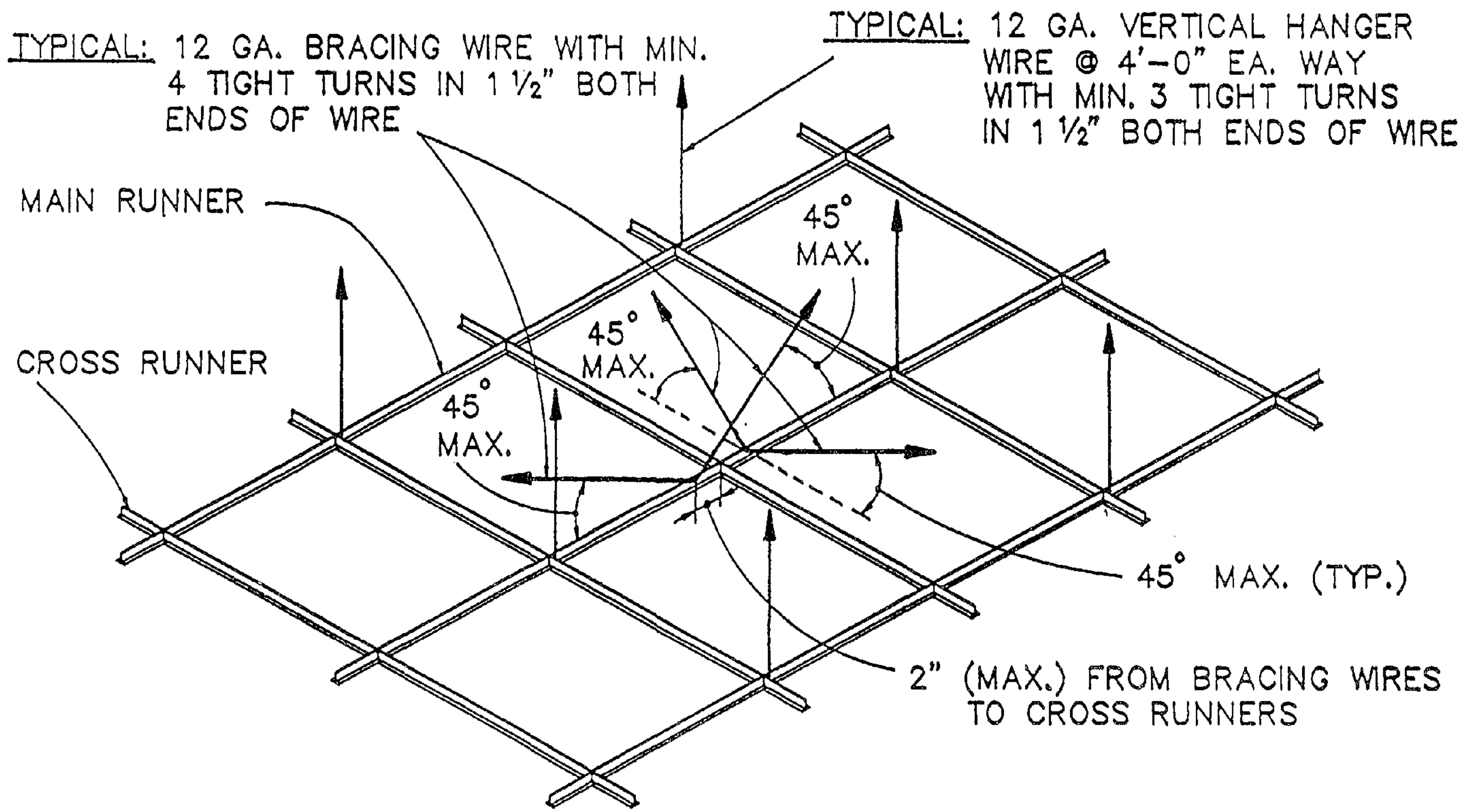


Figure 11.2 - Attachment of Acoustial Ceiling Grid

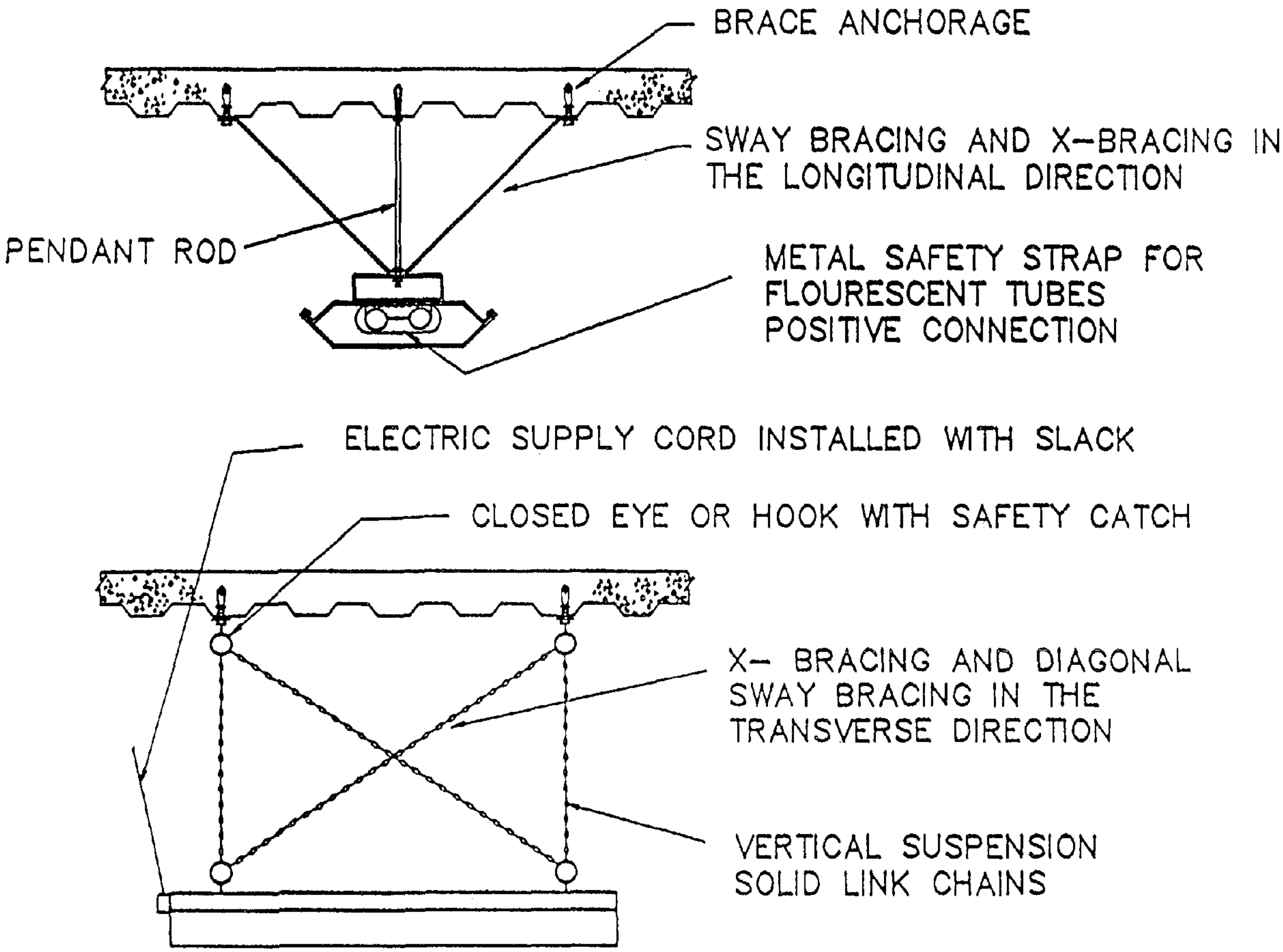


Figure 11.3 - Typical Anchorage of Suspended Lighting

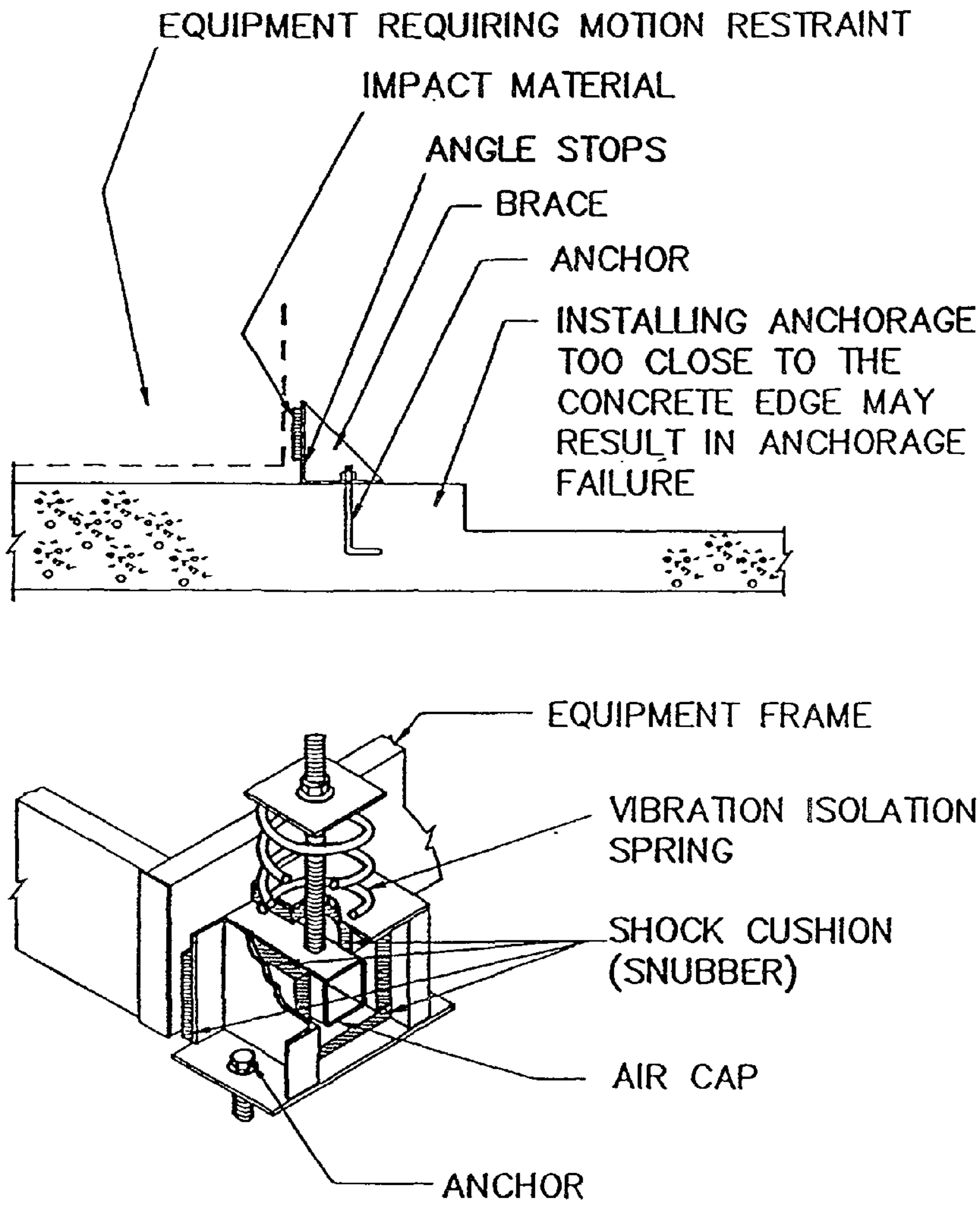


Figure 11.4 - Equipment Motion Restraint Systems

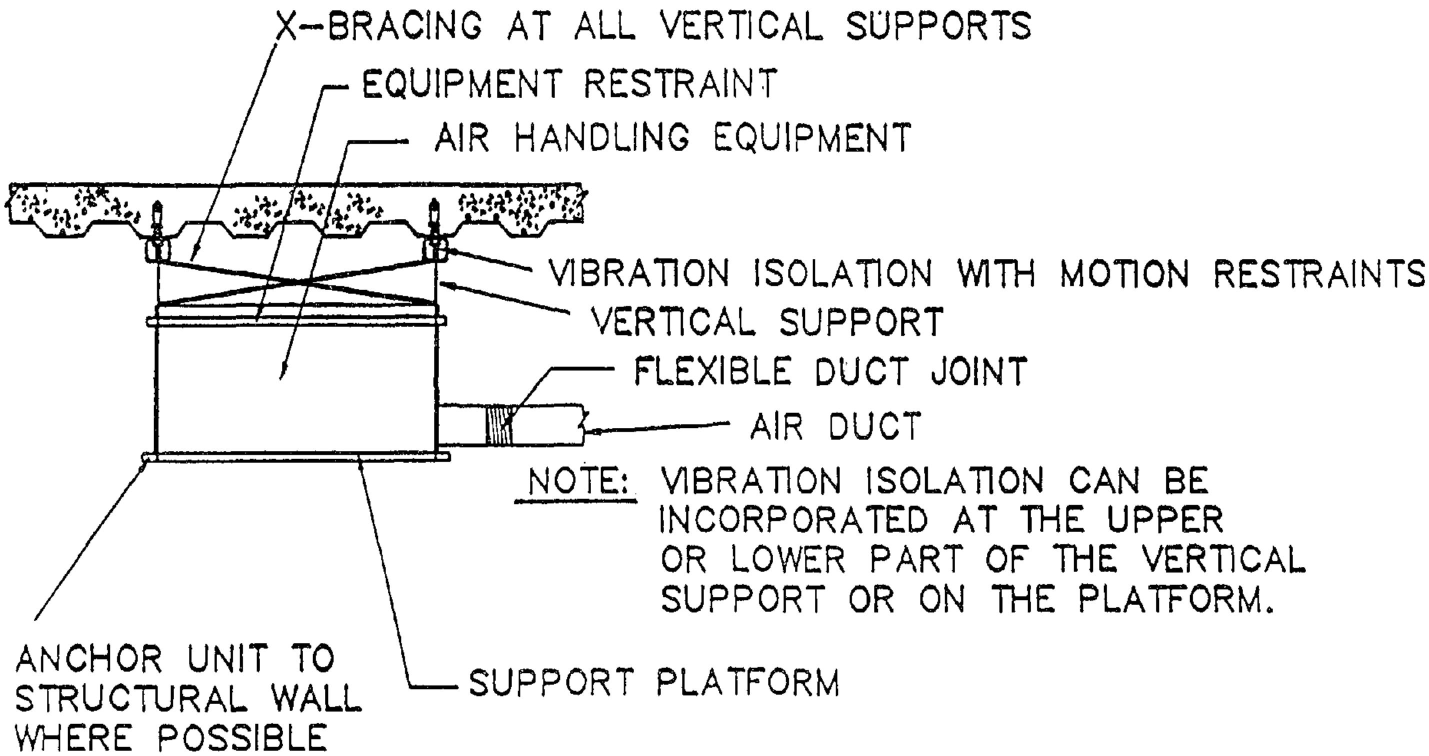


Figure 11.5 - Typical Anchorage of Air Handling Equipment

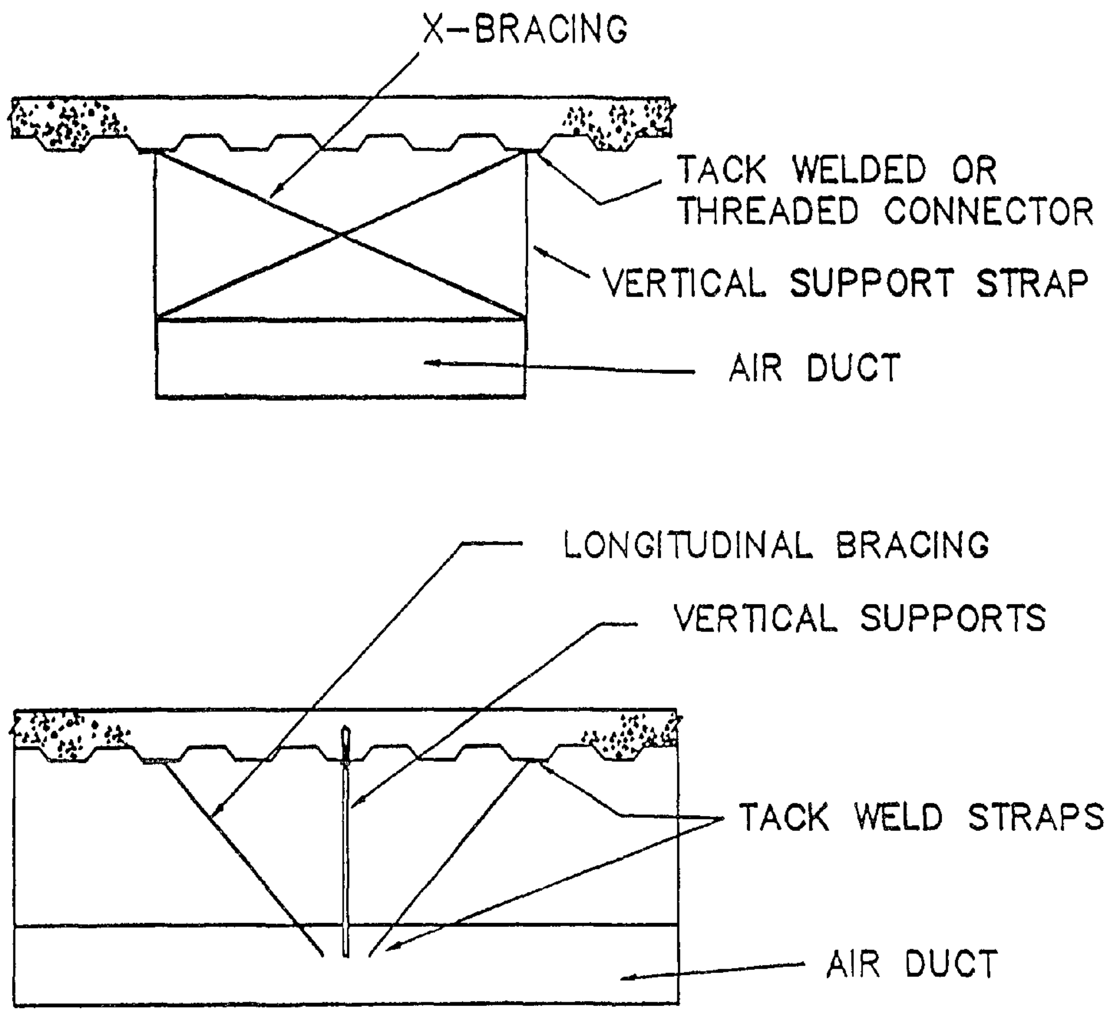


Figure 11.6 - Typical Suspension and Bracing of Ducts

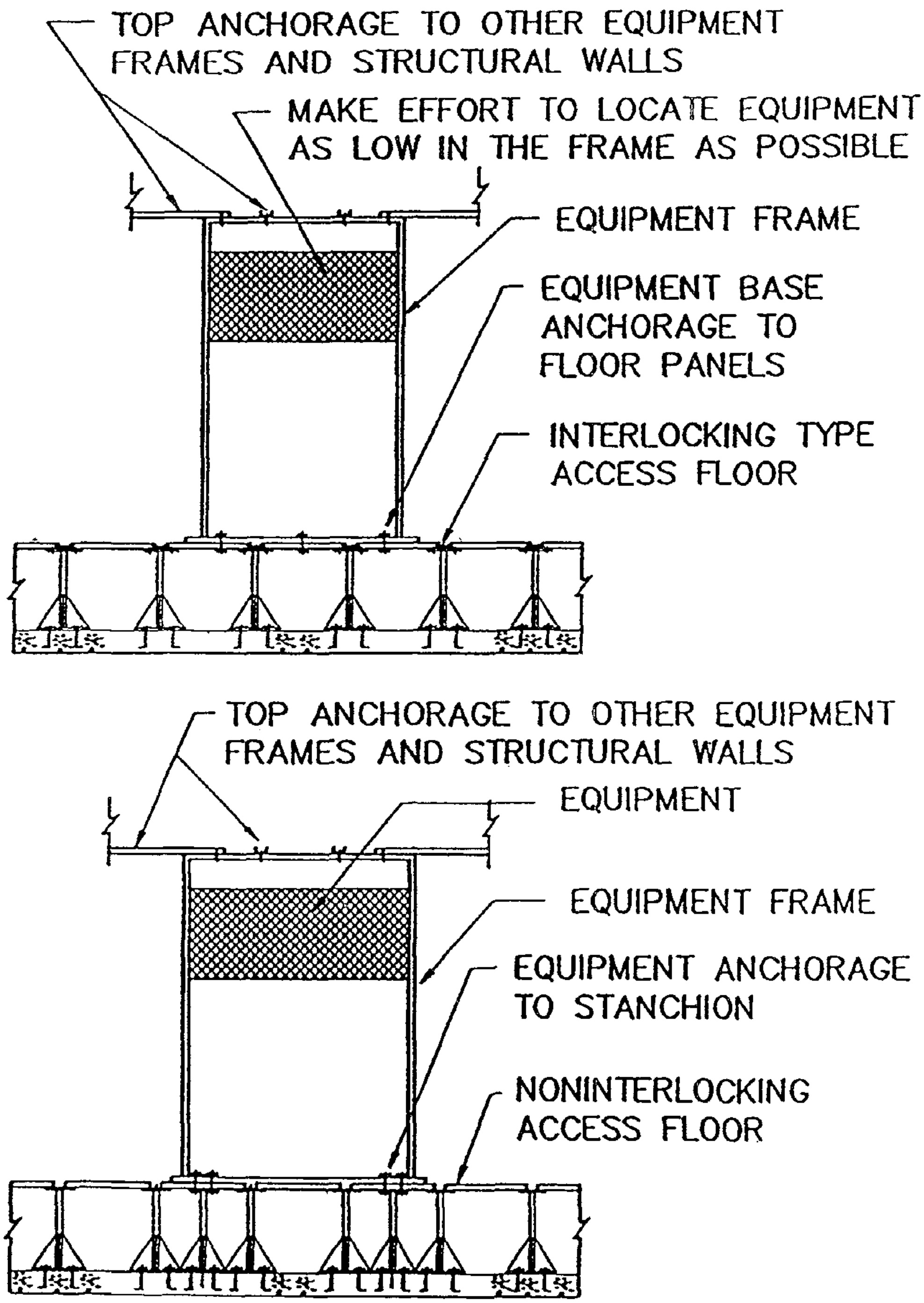
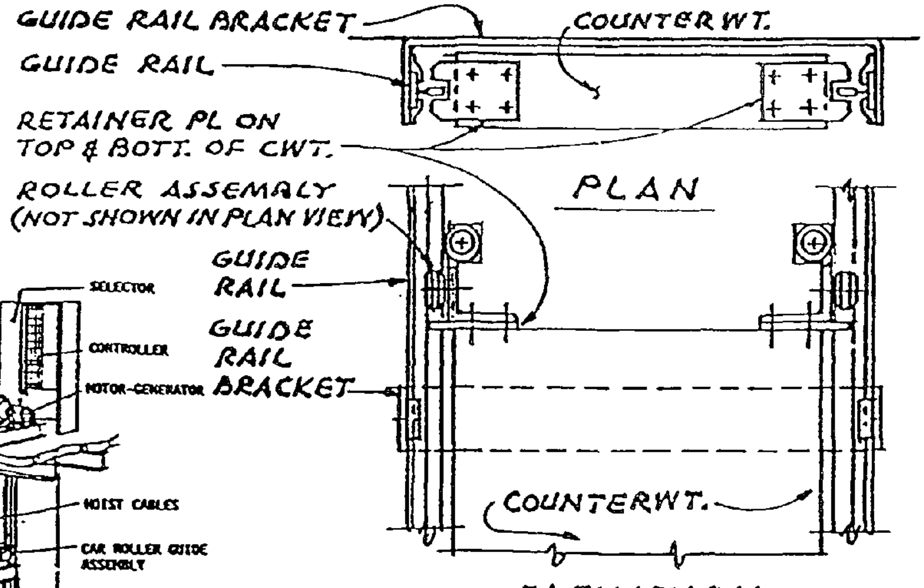
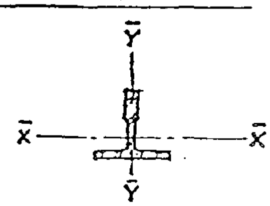
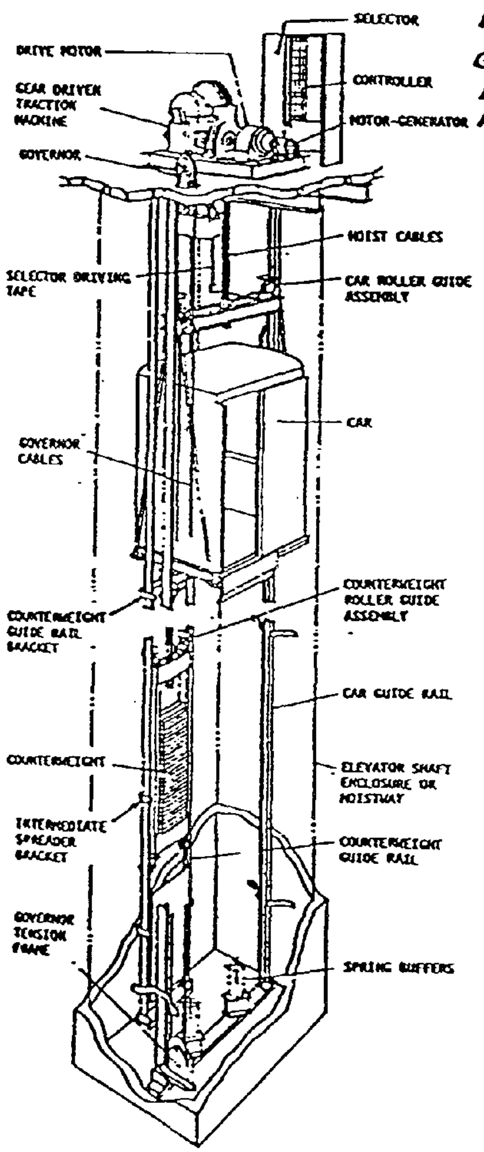


Figure 11.7 - Anchorage Details for Equipment
Supported on Access Floors



ELEVATION





TRACTION TYPE ELEVATOR

SECTION PROPERTIES OF ELEVATOR GUIDE RAILS

Nom. Wt. Per Foot	A	$\bar{y}$	I_x	S_x	r_x	I_y	S_y	r_y
8	2.40	.81	1.40	.86	.78	1.28	.73	.74
11	3.24	1.12	4.29	1.80	1.15	2.67	1.19	.90
12	3.51	1.08	4.49	1.85	1.13	3.61	1.45	1.01
15	4.41	.97	4.78	1.89	1.04	5.51	2.21	1.12
18.5	5.44	1.26	9.67	3.23	1.33	7.45	2.71	1.17
22.5	6.73	1.38	11.17	4.20	1.28	8.64	3.10	1.13
30	8.93	1.74	24.00	7.37	1.64	11.08	4.03	1.11

A = Area ($in.^2$)
$\bar{y}$ = Distance from back of rail to axis x-x (in.)
I_x = Moment of inertia about axis x-x ($in.^4$)
S_x = Section modulus about axis x-x ($in.^3$)
r_x = Radius of gyration about axis x-x (in.)
I_y = Moment of inertia about axis y-y ($in.^4$)
S_y = Section modulus about axis y-y ($in.^3$)
r_y = Radius of gyration about axis y-y (in.)

Figure 11.8 - Elevator Details

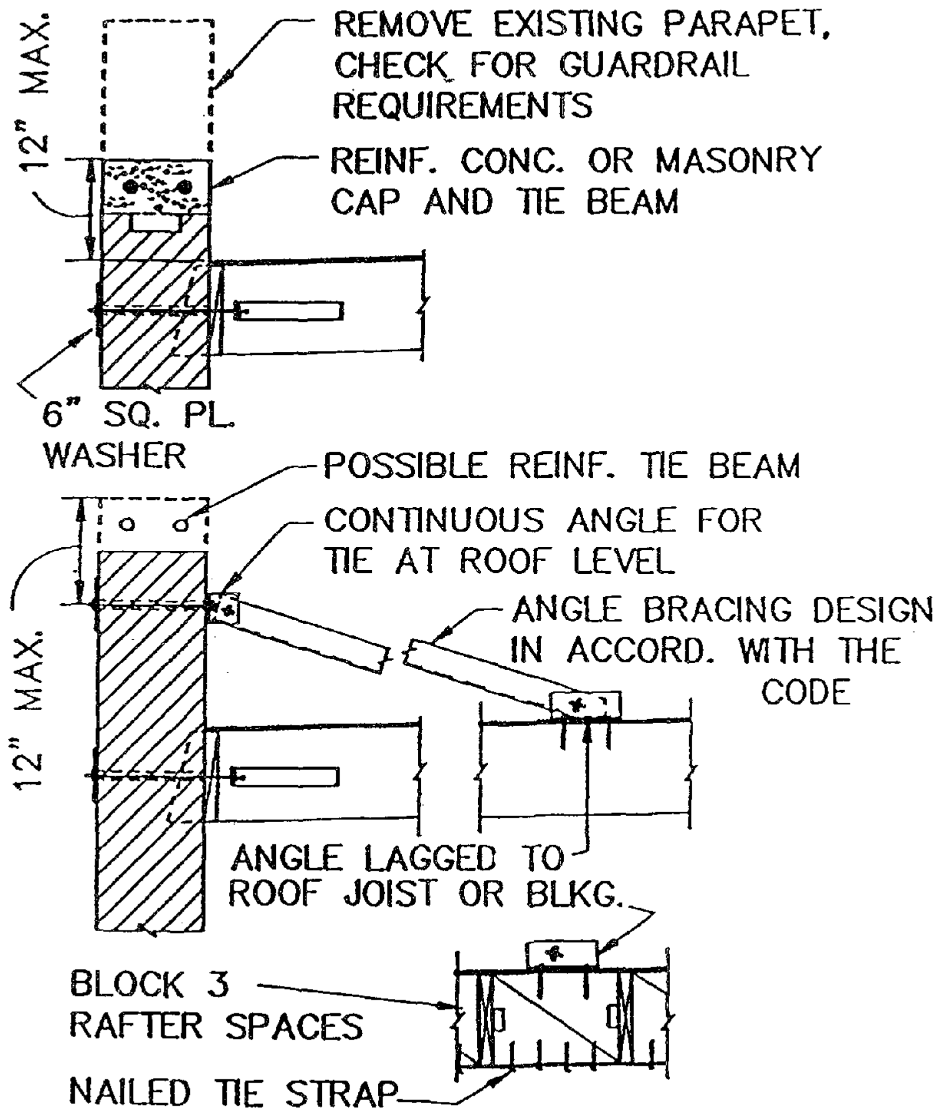


Figure 11.9 - Typical Parapet Anchorage Detail

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

ATC-14, titled "Evaluating the Seismic Resistance of Existing Buildings", was recently developed as a methodology which would be applicable nationwide. But, none of the major participants in the original project were practicing engineers from the Eastern United States. This fact coupled with the increasing awareness of Eastern seismicity resulted in a need to critically assess the applicability of the ATC-14 document to the Eastern United States.

The National Center for Earthquake Engineering Research (NCEER) is presently coordinating a five-year research plan which is designed to systematically study earthquake engineering topics. One of the major programs for the second year of this five-year plan was a topic titled "Existing Structure". As a portion of this program, NCEER funded a project to critically review ATC-14's applicability to Eastern United States construction. They contracted H.J. Degenkolb Associates, the San Francisco-based firm who served as the Subcontractor and primary author of ATC-14, to serve as the Principal Investigator for this review.

In conjunction with investigators at Cornell University, Degenkolb selected a review panel of five engineers from the Eastern United States who are knowledgeable in seismic design. During their review and subsequent project meetings, the project team identified a number of issues and topics where they felt that significant improvements could be made to ATC-14. These issues and topics became project tasks which were developed into recommended additions or revisions by the members of the project team. These recommendations, which are listed below, are detailed in this report:

1. A discussion of current NCEER projects which are studying topics which could provide results that would be useful to future editions of ATC-14. Future research topics which could improve ATC-14 are suggested.

2. A discussion of the present state of knowledge on Eastern U.S. Seismicity which occurred during a meeting with seismologists in conjunction with an NCEER sponsored conference on eastern earthquake hazards.

3. A description of the regional similarities and differences which exist between the Eastern and Western United States in seismic design and evaluation.

4. A collection of additional information which could be useful in a seismic evaluation. This information includes a list of historical documents on building construction, an expanded list of reference standards, a compilation of state code adoption status, and a list of earthquake damage data for Eastern United States earthquakes.

5. A major revision and expansion of the ATC-14 sections which provided the seismic evaluation procedure for buildings in regions of low seismicity.

6. A major revision and expansion of the ATC-14 Chapter on non-structural elements.

It is intended that these recommendations be considered for inclusion in any future revised editions of ATC-14, and the ATC-22/FEMA project which is currently developing a handbook for seismic evaluation which is based on ATC-14. This report will also serve as an excellent supplement to ATC-14, and will be especially useful for the seismic evaluation of buildings in areas of low seismicity.

Because of the volume of the recommended modifications, NCEER has decided to fund a follow-up project which will incorporate all of the information presented here with the original ATC-14 documents to generate a new document which is specifically intended for the seismic evaluation of buildings in regions of low seismicity. This report will provide a valuable tool for engineers performing these evaluations on buildings in the Eastern United States.

CHAPTER 9

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APPENDIX A

PROJECT MEETING MINUTES

NCEER PROJECT ON SEISMIC EVALUATION OF BUILDINGS IN THE EAST

MEETING MINUTES

DECEMBER 3 and 4, 1987, MEMPHIS, TENNESSEE

Attendees:	Peter Gergely	- Cornell
	Richard White	- Cornell
	Glen Bell	- Simpson, Gumpertz and Heger
	Charles Lindbergh	- The Citadel
	Warner Howe	- Gardner and Howe
	Chris Poland	- H.J. Degenkolb Associates
	James Malley	- H.J. Degenkolb Associates

I and III GOALS AND OBJECTIVES - Chris briefly reviewed the goals of this project: To critically review and improve ATC-14 to make it more applicable to buildings in the East. The input from the Review Panel of Eastern Engineers will be incorporated into proposed revisions which can be incorporated with the ATC-22 work. The Panel will form a working group that will develop the proposed revisions. All project meetings will occur in regions of lower seismicity so that the group can observe regional construction characteristics. Degenkolb is still waiting for a contract from Cornell. Once we complete our contractual agreements, we will send out individual contracts to the Panel Members.

II INTEGRATION OF THIS WORK AND NCEER PROGRAM FOR EXISTING BUILDINGS - Peter handed out a list of the Center's programs for the first two years. The "Existing Buildings" topic comprises one of three broad programs which compose the majority of the Center's funding in Year Two. Existing Buildings cover a broad range of research. One group includes testing of old styles of construction. Jacob Grossman will consult on older concrete construction details. There is a coordination meeting of the experimental researchers next week. Another important topic is the development of Expert Systems for seismic design. Fenves at Carnegie-Mellon is developing a long-range system which will result in a safety index. The Cornell group is developing its system based on expert opinion questionnaires. There will be a workshop in August at Cornell on Expert Systems. Another large area of work is on Ground Motion. Klaus Jacob at Lamont-Doherty is the lead researcher in this area. They are installing instruments and could assist us in determining the design ground motions.

Charles said that there is an immediate need to better define Eastern seismicity. He said that the State Geologists of South Carolina has initiated such a program for their State. Warner said that outside of the Memphis area there is not much definition of Eastern sources. Peter said that Lamont could assist us with any specific questions. Charles suggested that a meeting be set up in conjunction with the large convention being held in New York City next February to address these topics.

Glen Bell described the experience gained in developing the seismic provisions for the Massachusetts Building Code. Because of the sparse data, they had to determine an acceptable level of risk based on cost and societal expectations. As a result, using the 1755 Cape Ann Event, an MMI = VII was selected. This corresponds to approximately .12g. Since wind controls the force level in many cases, the provisions only affect the ductility of elements, resulting in fairly minor cost increases for new construction. They estimate the recurrence interval to be 2,000-10,000 years for this event.

There was another discussion of the need to update the maps. Warner and Charles stated that the Standard Building Code and the BOCA Code will include the maps.

Glen likes having the ability to use different recurrence intervals as prescribed in Chapter 3.

IV Chris then presented a set of slides taken from the seminar series to give the Panel some insight into the development of ATC-14. Some specific comments made during this discussion included the following:

Warner: There is an education problem with many Engineers who believe that the code seismic forces are the real loads.

Dick: Cornell is doing testing on short lap splices.

Glen: The rapid shear stress check in walls may not be representative since overturning is often the problem.

Dick: Deterioration is a larger problem in the East. Glen added that freezing is part of the cause.

Glen: The concept of separating the C/D ratios according to ductile, semi-ductile, non-ductile is a good one.

Glen: Untopped concrete plank floors on short bearings are typical for residental construction. Charles feels there should be a minimum thickness check for the topping slab.

V GENERAL DISCUSSION OF THE DOCUMENT - Glen feels that we should reference as many standards of practice as possible. Such documents as older books, which discuss construction techniques at the time can be very helpful. Older codes which may have been in effect at the time of construction can also be useful.

Glen also brought up our referencing the UBC on page 53 for the different materials. Other standards such as NDS, ACI, BOCA, SBC, etc., should also be included. Charles feels that we need to integrate all the applicable standards.

Dick White brought up the topic of connection adequacy. We need more figures and examples of different details with companion reference citations. We may want to list a set of required reference documents at the beginning of the document.

Glen brought up Table 4.10 which is referenced from ABK. He suggested that we add citations to specific standards which would allow the Engineer to use other values if he/she has better information. Charles added that the metal deck standards are hard to use and may give conflicting values.

Glen feels that we should discuss geotechnical aspects and foundation design in more depth. Two issues of concern are rubble foundation walls and pile caps without ties. Massachusetts Code has a first cut at liquefaction.

We should compare ATC-14 to the codes that are being used in various regions.

Discussion on Addition to Building Damage for Eastern Buildings:

Charles: Charleston has a record of an index of the response of all buildings during the 1886 earthquake. Chris suggested that he write a short paper for EERI Spectra to publish this data.

Dick: John Stevenson has information on damage caused by the Ohio earthquake. Dick will try to get a copy of this information.

Warner: Allen and Hoshall's report to FEMA on seven cities may include a Memphis building inventory. Warner will try to get a copy of the inventory.

VI DISCUSSION ON EACH CHAPTER -

CHAPTER 1 - INTRODUCTION

Page 1 - Glen said that since most structures in the East were not designed for any seismic forces, we should modify the first paragraph. Peter added that the existing conditions may vary significantly from the drawings due to variations in construction and deterioration over time.

Page 2 - Charles brought up that the phrase "used as a guide by Structural Engineers experienced in seismic design and analysis.." would exclude almost all Eastern Engineers. The ATC-22 Handbook may help with the definitions. Dick suggested that we add a discussion of the term "most probable large earthquake event".

Page 3 - We should add a paragraph on the state of the practice to this Chapter.

CHAPTER 2 - STATE OF PRACTICE REVIEW

Glen suggested that we add a brief historical discussion of when different codes started requiring seismic design and what the provisions included.

Dick mentioned that we may want to add the new Japanese procedure for evaluating steel buildings.

Page 7 - We should add references to Eastern earthquakes in paragraph 4.

Dick spotted an inconsistency between the questionnaire discussion and the summary on page 333 concerning ductility checks on connection details.

Glen and Charles think that it may be beneficial to discuss the issue of connection design responsibility.

CHAPTER 3 - SEISMIC LOADING CRITERIA

Dick feels that the discussion of duration should be expanded.

Glen stated that the attenuation differences between East and West should be discussed. Warner added that since much of the Eastern U.S. work is based on extrapolation, the discussion should point out where the recommendations rely on only limited information.

Charles feels that we should emphasize the use of microzoning where possible. This is not site-specific, but rather by areas. Warner added that the new BSSC maps will identify locations of maximum shaking at the center of the different contours. Charles again called for a small workshop in New York to better define the zonation, duration and attentuation characteristics. This may be coordinated through the Center.

Glen would like a cost-benefit discussion. Since in the East the recurrence interval is 2,000-10,000 years, the definition of life safety is more difficult. We should add a discussion of going beyond life safety for essential facilities.

Glen feels that it would be helpful to add a comparison of ATC-14 force levels to those of other common codes, similar to their paper on the Massachusetts Code. He also would like some commentary discussion on how we developed the R_w factors on page 55. Possibly in Section 4.4.3, paragraph 2.

Charles would like the addition of a discussion of distant earthquakes such as the Nuttli's ASCE paper. It is already included as A_v, but should be brought out in the discussion.

CHAPTER 4 - GENERAL METHODOLOGY FOR THE EVALUATION OF EXISTING BUILDINGS

Page 54 - Dick feels that the 2.5 factor in Eqn. 4.1 may not be high enough for Eastern buildings which are subject to more severe deterioration problems. We should also explain the difference between R and R_w in the second paragraph of Section 4.4.3.

Page 53 - Glen again stated the need to expand and integrate the referenced standards.

Page 44 - Charles feels we should add a check for minimum thickness of topping slabs. This can be incorporated into the existing statements.

Page 39 - Glen feels we should expand the discussion in the first paragraph to explain what we mean by "the basic elements of the lateral force resisting system".

Pages 40, 42, 43 - Glen mentioned that our discussion of deterioration needs to be expanded for the more severe Eastern environment. Freeze-thaw conditions for parapets and unbonded veneer, corrosion in coastal environments, water penetration of the building envelope, roof problems, snow loads, salt on garage decks, etc.

Charles added that we should alert the user to possible overloads due to change of occupancy during the life of the structure.

Dick questioned the applicability of using the same C/D ratios for low strength concrete and steel. Does it have similar ductility?

Page 52 - Charles states that in the coastal areas, there are many buildings in the flood plan which are placed on piles. This type of construction, which is also prevalent at waterfronts, may need to be considered in the documents. He also feels that we need to provide more direction on how to integrate two types of buildings.

Page 58 - Dick pointed out that #7, "Alternate Procedures" is vague. He also feels that in the rapid analysis procedure in Section 4.4.2, we should explain why the method is limited to six stories. It could also be stated in the title of the section to avoid confusion.

Page 40 - Glen would like to note the historical references which could be useful. We will add a list of such documents to the references in the document.

Page 69 - Glen feels that we should explain the philosophy behind using I = 1.5 for containers of toxic or explosive materials.

Page 42 - Charles suggested that we expand "location of adjacent structures" to include the condition and potential for damage to the building being evaluated.

CHAPTER 5 - SEISMIC EVALUATION OF WOOD FRAMED BUILDINGS

Dick asked if the level of connection between elements of wood structures differed between East and West. A discussion of stud wall anchor details indicated that there are differences.

Glen suggested that we add statements 5.6.5, 5.6.8, 5.6.11, and 5.6.13 to the Low Seismicity list. The basic feeling was that the generic statements in the low seismicity areas need to be more specific.

Dick and Glen felt that in Statement 5.5.2, nonredundant but ductile could be acceptable.

Page 89 - Statements 5.5.2 and 5.5.4 have not been updated to match the other sections.

Page 90 - Dick pointed out that the wording at the start of Section 5.6 should match that of Section 5.5.

Page 91 - Glen wants to add a check for bug infestation caused by contact between wood and soil.

CHAPTER 6 - SEISMIC EVALUATION OF STEEL FRAMED BUILDINGS

Section 6.1 - Moment Resisting Frames --

Page 98 - For buildings over a few stories, a review more detailed than that required by Statement 6.1.5.1 may be necessary.

Glen suggested that we add Statements 6.1.6.4, 6.1.6.5, 6.1.6.6, 6.1.6.8, 6.1.6.11, 6.1.6.13, and 6.1.6.16 to the Low Seismicity list. Statement 6.1.6.7 should be modified and 6.1.6.15 should apply for buildings over two or three stories. For tall buildings, the ductility demands may be just as large due to reduced capacity of the elements. This also applies for Section 6.2.

Section 6.2 - Braced Steel Frames Buildings --

Glen suggested that we add Statements 6.2.6.1, 6.2.6.3, 6.2.6.4, 6.2.6.5, 6.2.6.12, 6.2.6.13, 6.2.6.14, and 6.2.6.16. Statement 6.2.6.10 needs to be added to the Low Seismicity list to a lower ductility criteria. A figure is needed for Statement 6.2.6.8. The rapid stress check for braces should be reviewed and given a number.

Section 6.3 - Light Metal Buildings --

Charles brought up the fact that many light metal buildings have heavy exterior masonry walls which are not infilled, for two, three, or even eight-story buildings. We should add this to the Performance Characteristics. He also stated that heavy mechanical equipment which could be suspended from the roof may cause local distress in the diaphragm. This could also be added to Chapter 11.

Glen feels that Statement 6.3.6.11 should be covered in Chapter 11. He would like us to expand the discussion of exterior wall systems. The statements which should be added to the Low Seismicity list are 6.3.6.3, 6.3.6.4, 6.3.6.5, 6.3.6.6, 6.3.6.7, 6.3.6.8 and 6.3.6.10.

Section 6.4 - Seismic Evaluation of Steel Frame Buildings with Cast-in-Place Walls --

Dick asked if the title should be "Concrete Shear Walls" rather than "Concrete Walls".

Glen thinks that a rapid check of shear and overturning should be added for areas of low seismicity. He said that overturning can be the limiting factor when the gravity loads to the wall are low. Glen suggested that we add Statements 6.4.6.3 through 6.4.6.9, 6.4.6.12 through 6.4.6.15, 6.4.6.17, 6.4.6.19, and 6.4.6.20 to the Low Seismicity list. The H/D = 4 in Statement 6.4.6.10 may need revision.

Chris stated that we may just have one set of Statements with special requirements for highest seismicity.

Charles would like us to add exterior wall courses to Statement 6.4.5.6.

Dick said that any 1mm crack may be important.

Section 6.5 - Seismic Evaluation of Steel Framed Buildings with Infilled Walls of Unreinforced Masonry --

Page 143 - Clarify the A_w is the gross area of the wall in the rapid stress check. Some stress check is useful for buildings over three stories to get more uniformity in the evaluations.

Glen suggested that we add Statements 6.5.6.4 through 6.5.6.14 to the Low Seismicity checklist. In 6.5.6.12, the bonding should be investigated before requiring bracing.

Glen asked if there should be a region of EPA $\leq$ .05g for low seismicity. Charles suggested that we may want to include Professor Winfred Carter of Florida State University into our group.

Other Issues - 1) We will not be able to support Bob Hanson's research with our grant. 2) Peter stated that we can identify areas for testing research priorities for the Center. 3) Charles suggested that Chris present ATC-14 to a group of thirty South Carolinians at their March meeting in Columbia. We may have our next review meeting in conjunction with the State meeting.

CHAPTER 7 - SEISMIC EVALUATION OF CAST-IN-PLACE CONCRETE BUILDINGS

Section 7.1 - Concrete Moment Frame Buildings --

Glen questioned Statement 7.1.5.7 where we only check that the shear strength is greater than the moment strength of the frame columns. He said that the moment capacity almost always controls, but they still require sufficient strength.

Glen suggested that the following Statements be added to the Low Seismicity list: 7.1.6.2, 7.1.6.4, 7.1.6.5 through 7.1.6.11, 7.1.6.19 through 7.1.6.22. Statements 7.1.6.13, 7.1.6.15, 7.1.6.16, and 7.1.6.18 should be included with less restrictive requirements such as the Massachusetts Code or ACI Semi-Ductile Requirements. In 7.1.6.9, add a Pass/Fail Criteria. In 7.1.6.10, add a minimum topping slab thickness and connection requirement.

Peter feels that the .005H drift limit may be too strict. This value should be tested.

Section 7.2 - Concrete Shear Wall Buildings --

The comments in this section are similar to those for Section 6.4. Dick added that the 2.0 in Statement 7.2.6.15 should be changed to 2.

Section 7.3 - Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry --

The comments in this section are similar to those in Section 6.5. We should add Statements on the quality of infill construction in these sections.

Glen had a general comment on two floor systems; tile arches and brick arches which could be hazardous. We can get information on these systems from Eastern European earthquake reports such as the AISC report on the Skopje Earthquake in Yugoslavia. We should add this to our descriptions and into the Statements. This system was used in the Northeast between 1900 and 1920.

CHAPTER 8 - SEISMIC EVALUATION OF BUILDINGS WITH PRECAST CONCRETE ELEMENTS

Section 8.1 - Tilt-Up Buildings --

Page 191 - Reword to say "strength and ductility" in the introductory paragraph of the Performance Characteristics.

In this section, similar concerns were raised about moving Statements into the Low Seismicity list. It was suggested that we show details which will make it easier to distinguish between ductile and non-ductile details. Move Statement 8.1.6.7 into the Low Seismicity list.

Section 8.2 - Precast Concrete Frame Buildings --

Comments are similar to R/C frames. We should add a discussion of the freeze-thaw problems for parking structures. Move Statements 8.3.6.16 and 8.2.6.6 into the Low Seismicity List.

CHAPTER 9 - SEISMIC EVALUATION OF REINFORCED MASONRY BUILDINGS

We were asked to consider where cast-in-place concrete floor diaphragms would be placed in this Chapter. We will clarify that they should be considered in Section 9.1.

Section 9.1 - Reinforced Masonry Bearing Wall Buildings with Wood on Metal Deck Diaphragms --

Glen suggested that we move Statement 9.1.6.10 into the Low Seismicity section as well as those discussed previously.

Section 9.2 - Reinforced Masonry Bearing Wall Buildings with Precast Concrete Diaphragms --

Glen feels that Statement 9.2.6.8 should be moved into the Low Seismicity list with the other items suggested in other sections.

CHAPTER 10 - SEISMIC EVALUATION OF UNREINFORCED MASONRY BEARING WALL BUILDINGS

Chris stated that ATC-22 will develop a section for areas of low seismicity for this type of building.

Charles asked about how footings were to be covered. Brick footings with deteriorated, eroded or weak stone could be a problem. Rubble footings and basement walls are also common.

CHAPTER 11 - SEISMIC EVALUATION OF NONSTRUCTURAL ELEMENTS

Glen said that the wording in the introductory paragraphs leads the reader to think that all items in this Chapter are concerned with damage control only. We will clarify this introduction.

Glen suggested that we designate the life safety items in the list of Performance Characteristics (Section 11.1).

Charles said that our list may not be detailed enough for the evaluation of critical facilities. Chris said that we may add some discussion of hospitals in the introduction of this Chapter. Charles also suggested that we add more descriptions of what to look for, perhaps with figures.

Glen stated that Sections 11.1.11 and 11.2.10 need expansion to cover all types of wall systems. We should also flag deterioration of anchorages.

In Section 11.2.9, we should check on the requirements for cable restraints.

CHAPTER 12 - EXAMPLES OF USE OF THE METHODOLOGY

Chris stated that ATC-22 will perform a large number of trial evaluations. We are not planning to evaluate any test buildings in this project.

These Minutes summarize the notes taken at the two-day meeting in Memphis, concerning possible revisions to the ATC-14 document. As a result of this discussion, we have developed an Action Plan for Work to be Completed in the Second Year. This plan is presented separately. If you have any questions, comments, additions or modifications to these Minutes, please contact us as soon as possible.

Respectfully submitted,

H. J. DEGENKOLB ASSOCIATES, ENGINEERS

James O. Malley

James O. Malley

JOM/dq

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NCEER PROJECT ON SEISMIC EVALUATION OF BUILDINGS IN THE EAST

MAJOR TASKS TO BE COMPLETED IN THE SECOND YEAR

The following tasks were generated as a result of the December meeting in Memphis. Along with each task, there are designated member(s) of the group who will be primarily responsible with addressing that topic. We are hoping to obtain your input to these items by the end of January, 1988, so that we can maintain the proposed project schedule.

TASK #1 - Liaison with other NCEER Projects

- Dynamic Analysis and Testing (Semi-Rigid Connections, e.g.)
- Expert Systems
- Evaluation of Buildings in New York
- Ground Motion

Peter Gergely will keep us informed as to the direction and interaction which this group can make with these projects.

TASK #2 - Update A_A and A_V maps for the Eastern U.S. based on best information available.

- Alert researchers at Lamont-Doherty to the need to develop specific map type information.
- Review and update the recurrence interval modifiers for use along the East Coast.
- Review and update the effects of distant earthquakes and durations.
- Convene a workshop to occur in conjunction with the large meeting in New York in February 1988 to discuss these issues.

Peter Gergely and Charles Lindbergh will contact Klaus Jacob and Walter Hays to set up the workshop meeting.

TASK #3 - Develop a list of historical documents which discuss construction techniques or include other information useful in performing structural evaluations. (Kidder's Construction Guide for Superintendents, e.g.) Identify the regions in which they apply.

All members of the group should contribute to this list, as possible.

<u>TASK #4</u> - Develop a list of all model building codes for adoption in the East. Indicate the year of adoption and editions with seismic provisions. Compare the force level to ATC-14.

All members of the group should contribute to this list, as possible.

<u>TASK #5</u> - Expand the list of current standards referenced for use in calculating allowable stresses. List any reservations or problems with using these documents.

All members of the group should contribute to this list, as possible.

<u>TASK #6</u> - Develop a recommended list of reference material needed for using ATC from the results of Tasks #4 and #5 above.

All members of the group should contribute to this list, as possible.

<u>TASK #7</u> - Add figures to the text of ATC-14, as appropriate. Degenkolb will develop the figures suggested during the meeting. Any additional suggestions for figures can be submitted by all members of the group.

<u>TASK #8</u> - Prepare a discussion of the evaluation of site effects including liquefaction, landslide, and faulting. These items should be discussed in terms of available regional studies.

Glen Bell will send Degenkolb a copy of the Massachusetts Code requirements. Degenkolb will develop the discussion.

<u>TASK #9</u> - Identify Eastern earthquake experience and add to appropriate sections.

All members of the group should collect any references of such available information. Degenkolb will develop the additions.

<u>TASK #10</u> - Rewrite sections on Low Seismicity. Expand and differentiate between high and lowrise (3 stories) buildings.

Degenkolb will develop this task from the comments generated during the meeting.

<u>TASK #11</u> - Develop Statements for deterioration due to East Coast environmental effects, such as bug infestation, freezing and thawing, corrosion, etc.

Glen Bell will prepare these Statements, Concerns, and Procedures in language similar to that used in the ATC-14 document.

TASK #12 - Develop Statements related to wharf structures. Determine how these statements can be incorporated into ATC-14.

Charles Lindbergh and Warner Howe will supply information on these issues to Degenkolb. Degenkolb will prepare the Statements.

TASK #13 - Develop Statements related to exterior masonry walls which are braced by vertical load carrying frames of steel or concrete.

Glen Bell will develop these Statements.

TASK #14 - Review Statements related to overturning in concrete shear walls.

Glen Bell will provide Degenkolb with his studies for buildings in New England. Degenkolb will use this information in reviewing the applicability of these Statements.

TASK #15 - Identify the most critical research needs for the different building types.

All members of the group should contribute to this list, as possible.

TASK #16 - Review Statement 7.1.5.7. Review damage reports to validate any modification.

Glen Bell will provide Degenkolb with his studies for buildings in New England.

TASK #17 - Incorporate semi-ductile requirements for areas of low seismicity in concrete frame buildings. Use Massachusetts Code and ACI requirements, as appropriate.

Glen Bell will provide Degenkolb with the Massachusetts Code provisions. Degenkolb will develop the Statements.

TASK #18 - Review the appropriateness of the .005H drift limit for all building types.

Peter Gergely will provide Degenkolb with his information on walls and Mete Sozen's work on frames.

TASK #19 - Develop Statements for tile arch floors, using the information provided in the AISC publication on the Skopje Earthquake.

Glen Bell will develop these Statements.

TASK #20 - Develop Statements for brick and rubble stone footings. Determine the importance and applicability to regions of different seismicity.

Glen Bell and Charles Lindbergh will provide Degenkolb with information on these subjects. Degenkolb will develop these Statements.

TASK #21 - Expand the nonstructural descriptions to provide more information to the user.

Degenkolb will perform this task.

TASK #22 - Expand Chapter 11 to include all wall systems used in the Eastern United States.

Glen Bell will expand this Section.

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MEETING MINUTES

NCEER WORKING GROUP ON EASTERN SEISMICITY

THURSDAY, FEBRUARY 25, 1988

NEW YORK CITY

Attendees:

Peter Basham
Michel Bruneau
Peter Gergely
Robert Ketter
Warner Howe
Klaus Jacob
Jim Malley
Chris Poland
Leonardo Seeber
J.P. Singh
Paul Somerville
Larry Soong
Carl Turkstra
Daniele Veneziano
Richard White

I Overview of the NCEER Project -

Chris Poland described the purpose of the NCEER project for which H.J. Degenkolb Associates is acting as the Principal Investigator. The project is intended to critically assess the applicability of and suggest improvements to ATC-14 for buildings in the Eastern and Central United States. During the initial project meeting, Professor Charles Lindbergh of the Review Panel suggested that there is a great deal of recently developed information on Eastern seismicity which has not been incorporated into the maps provided in ATC-14. He suggested that our project convene a meeting during this Conference to review Chapter 3 of ATC-14 with some of the seismologists who have been most involved in studying Eastern seismicity. Chris handed out a letter from Charles which describes the situation in South Carolina and the Southeast. Some of the issues which are to be addressed include the possibility of updating the A_a and A_v maps to reflect the best information available, the latest information on recurrence intervals, the effects of distant earthquakes and duration and areas where there are gaps in our present knowledge.

II General Discussion of the Seismic Zoning Maps -

Peter Basham stated that there is a need for a commonly accepted zoning map. The maps in ATC-14 are based on Algermissen and Perkins work in 1982, but more recent maps are possible based on new source models for the Eastern United States. The EPRI work has developed the best available model for Eastern seismicity. But, since the study was done on a site-specific basis, no map has been developed from the EPRI model. Klaus Jacob agreed that the EPRI model might be appropriate for developing a new map.

Klaus stated that he has not seen the basis of either Algermissen and Perkins or the EPRI work.

Paul Somerville stated that the EPRI study consisted of six separate teams which developed source models. The study never aggregated the results into a single map. Paul feels that the results would be significantly different from the ATC-14 maps in some areas.

Klaus described some of the areas where he suspected differences would occur. He said that based on recent Canadian studies, Maine looks too low. He added that there may not be any difference between Ohio and Indiana. He added that having three consecutive 0.1g contours is confusing.

A discussion of the letter prepared by Charles Lindbergh of the Citadel addressed the issue of local changes to the maps which result from microzoning techniques. While the group felt that the techniques were valid, it would be better to address the entire region as part of a larger project to develop a new map. It would be difficult to call for local changes except as part of a larger review of the entire region. Some changes to the area around Charleston could be expected as a part of this larger study.

Warner Howe stated that the 1988 revised NEHRP provisions employ the same maps as ATC-14, with contours and midpoints for interpolation added. A map for the 250-year earthquake is also included.

Chris suggested that we retain only the 475-year return period maps in an effort to be consistent with other design criteria. The members of the group agreed.

J.P. Singh stated that the USGS is convening a committee to hold three workshops to upgrade the present maps. J.P. added that A_v is low in Charleston and in Maine, since the damage is related to long period motion. Peter Basham stated that J.P. may be correct, but it would be difficult to call for changes to small areas. Larry Soong felt that it would be more appropriate to study the entire area.

Bob Ketter worried that if an NCEER-sponsored group developed a new map, it may make enemies. Paul Somerville and Peter Basham do not feel that such a project would create any enemies since the EPRI approach is a new procedure. Paul feels that the EPRI model is a good one since it quantifies the uncertainties, making it more useful to engineers.

J.P. related that the Algermissen and Perkins work is conservative in relation to attenuation. He said that three USGS-sponsored workshops are planned for the next eight months beginning this summer to address such topics as attenuation, applicable parameters, and new models. The ultimate goal is to develop a new set of national maps in five years.

Chris asked how long it would take to develop a map using the EPRI model. Klaus felt that the only change would be at the Maine/Canada border. Paul feels that there would be significant changes. He also stated that the product would be more justifiable since they are aggregate results. Daniele Veneziano stated that it would only take a few months to get PGA and the other parameters. He is concerned that the probability gets dropped after determining A_a and A_v. If spectral levels were mapped directly, a different result would occur due to the attenuation model. Paul stated that EPRI is doing thirty sites in six months, which shows that the work can be done quickly.

Daniele suggested that three parameters be combined with the three soils types for a total of nine maps. He said that if we include the uncertainty in the amplitude and soil factors, the results will be higher, but one step beyond the EPRI work. Chris asked how the equations for the Equivalent Lateral Force Procedure (page 62 of ATC-14) would be modified to incorporate this. Daniele said that another term would be added to these equations to utilize the nine maps.

J.P. stated that USGS could use the results of a map developed using the EPRI model in updating the present maps. It was agreed that it would be appropriate for NCEER to initiate a project to develop a map using the EPRI model.

III <u>Discussion of the Recurrence Interval Modification Procedure in ATC-14</u> -

Klaus feels that there is not enough documentation presented in the discussion in Chapter 3.

Peter Basham and Paul stated that there is information available to develop curves for differing recurrence intervals exactly rather than using the chart presented in Figure 3.8 of ATC-14.

J.P. said that Neville Donovan's approach was to use the Newmark-Hall factors to advance the basic spectral construction procedure.

Daniele stated that we may want to provide more information which allows the engineer using the document to decide on an appropriate recurrence interval. Chris feels that there may already be too much information to use properly. Carl Turkstra added that the load factors are based on an assumed recurrence interval, changing the recurrence interval may make the usual load factors inappropriate.

Daniele said that it may be necessary to use site specific studies to justify using different recurrence intervals. He said that it would be useful to map PGA for two or more different return periods. If the resulting maps are the same, Neville's approach is justified.

Carl added that the requirements for existing buildings are less than for new buildings. Klaus stated that this has been accounted for by using the mean rather than the one -signa amplification factors for the definition of the response spectrum as presented by Newmark and Hall.

Klaus stated that since the document does not present all the facts behind the development of these curves, it is hard to give a professional opinion. He suggested to "GO USE IT". Chris said that we will suggest that the information presented in the chapter be better documented, possibly through a separate technical paper. The ATC-22 project may also address this issue.

Daniele said that using 475-years or allowing a decision on the appropriate recurrence intervals is a central issue. Stated guidelines and a more precise rationale are needed. The 475-year number may be sufficient for California, but 1,000 or 5,000 years may be more appropriate for other areas.

IV Other Issues - Duration, Distance, etc. -

Paul stated that the EPRI work does not specifically address distance and duration. He feels that the best empirical information is related to long periods and distance. There is a need to check how EPRI incorporated this issue into their work. Paul stated that the EPRI work focused on distances less than 100 KM, but there is data available for longer distances.

Klaus stated that Lamont-Doherty will begin work on these issues. Klaus also stated that Gail Atkinson's studies with constant probabilities is good work. He added that ATC-14 implicitly uses this assumption.

Hopefully, the distance and duration issues can be folded into the proposed ground motion studies.

These Minutes summarize the notes taken during the NCEER Working Group Meeting on Eastern Seismicity. A set of Review Comments and Recommendations which resulted from the discussion at this meeting will be included in the work product of the present NCEER project. A draft of these Review Comments and Recommendations are presented separately. If you have any questions, comments, additions, or modifications to these Minutes, please submit them to us in writing by April 15, 1988.

Respectfully submitted,

H.J. DEGENKOLB ASSOCIATES, ENGINEERS

James O. Malley

JOM/dq

4/1/88
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REVIEW COMMENTS AND RECOMMENDATIONS RESULTING FROM NCEER WORKING GROUP MEETING ON EASTERN SEISMICITY

On February 25, 1988, a group of fifteen engineers and seismologists met to discuss the latest developments in the study of the seismicity of the Eastern United States. The group met specificially to discuss the seismic loading criteria presented in Chapter 3 of the ATC-14 document. This document, titled "ATC-14 - Evaluating the Seismic Resistance of Existing Buildings" is presently being reviewed for its applicability to the Eastern United States through an NCEER-sponsored project directed by H.J. Degenkolb Associates. The Review Comments and Recommendations which resulted from this meeting will be included as part of the work product for this project.

Major Review Comments -

1. The seismic zoning maps presented in the document are those developed by Algermissen and Perkins in 1977 and updated in 1982. Through the results of the EPRI work on source modeling, a great deal more information is presently available on Eastern seismicity. The EPRI model could be used to develop an entirely new seismic zoning map for the Eastern United States. These maps should not be altered in local regions because of the need to reconsider the entire Eastern region.

2. More recent information could cause significant modifications to some areas of the present ATC-14 maps. These areas include the following:
    a) Maine, near the Canadian Border
    b) Ohio
    c) Parts of South Carolina

3. A more explicitly probability-based procedure which includes the uncertainties in all the parameters could result in a more rational basis for determining the seismic loading. This would provide the engineer with more information on which to base his decisions.

4. The 475-year return period as the basis of the evaluation should be retained in order to be consistent with other design criteria. This return period may not be the most appropriate for other areas of the country.

5. There is a body of recently developed information on the effects of distance and duration of Eastern United States earthquakes which could be incorporated into this chapter. Lamont-Doherty is about to begin work on these issues.

6. As presented, Chapter 3 of ATC-14 does not present all of the background information which was used to develop the recommended procedures.

Recommendations -

1. NCEER should fund a study to develop a set of seismic zoning maps for the Eastern United States using the EPRI source model. The results of this work should be incorporated with the present USGS project which is updating the existing maps. A decision concerning which (and how many) parameters should be mapped should be done through a coordinated effort of engineers and seismologists.

2. The EPRI model should also be used to study the effects of differing recurrence intervals on the seismic zoning maps. If the form of the maps do not change for different recurrence intervals, an approach similar to that presented in Figure 3.8 of ATC-14 may be appropriate.

3. The effects of distance and duration should be incorporated into the studies recommended above. Lamont-Doherty should coordinate work on these issues.

4. Complete documentation of the procedures used to develop the information presented in Chapter 3 of ATC-14 should be published. This documentation could be in the form of a technical paper.

04/01/88
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NCEER PROJECT ON SEISMIC EVALUATION OF BUILDINGS IN THE EAST

2ND PROJECT MEETING - MEETING MINUTES

JUNE 28 AND 29, 1988 ARLINGTON, MASSACHUSETTS

Attendees:	Peter Gergely	-	Cornell
	Richard White	-	Cornell
	Glen Bell	-	Simpson, Gumpertz and Heger
	Charles Lindbergh	-	The Citadel
	Chris Poland	-	H.J. Degenkolb Associates
	James Malley	-	H.J. Degenkolb Associates
Absent:	Warner Howe	-	Gardner and Howe

I. **GOALS AND OBJECTIVES** - Chris welcomed the Project Team and began the meeting by discussing the intended Work Product that this project would generate. He stated that the product could either consist of a set of errata, or a stand alone revision to the original ATC-14 document.

Dick White said that they did not expect a stand-alone revision, since there is not enough budget to support such an effort.

Peter Gergely added that since the ATC-14 document is presently being revised in another ATC project for FEMA, a full revision is not warranted. Peter would like this report to be more than just a set of errata, it should be readable. Chris related that the ATC-22 project is just about to complete the trial evaluation phase, with a project meeting scheduled for July 22 that will include a presentation of our recommendations. Peter said that a sufficient goal for this project is to provide input to the ATC-22 revision. Chris then proposed that the report include a discussion of the major topics at the beginning of the report, followed by a list of the specific changes.

II. **GENERAL DISCUSSION** - Charles then began a discussion of the differences between the East and West in regards to earthquakes and earthquake-resistant design. He listed three major differences:

1) Seismicity - This topic was addressed at the meeting during the Conference in New York last February, and is the subject of ongoing projects funded by NCEER and others.

2) Training - The state of knowledge is growing in the East now, with motivators such as the Standard Building Code requiring seismic design. The level of expertise required should be at the level of the professional engineer. We should not require specialized knowledge. Glen stated that Massachusetts had a series of seminars on seismic design which were sponsored by ASCE. Charles added that a five year program is being developed for the Southeast to increase knowledge through a set of workshops.

3) Architecture and Design - There is presently not much information on the basic differences in the structures between the East and the West. The age of construction is one major difference.

III. LIQUEFACTION - Charles brought up the topic of liquefaction, which was a widespread occurrence in the Charleston Earthquake of 1886. He stated that South Carolina now has a liquefaction map which has caused this hazard to be viewed as the only possible cause of damage. Charles feels that we need to add a discussion of liquefaction to put it into perspective with ground shaking in terms of building damage. He likes the ATC-13 discussion which presents the basic information on this topic. He feels that we need to prepare a similar discussion for this report in order to alert the evaluating engineer, describe the types of building damage which can result, and direct the analysis to a Geotechnical Engineer when appropriate. Charles also feels that regional maps of liquefaction potential would also be useful, though these are beyond the scope of this project.

IV. DISCUSSION OF THE PROJECT TASKS

TASK #1 - Liaison with Other NCEER Projects

Peter reported on a number of other NCEER Projects which could provide information useful to this project.

1) Lightly Reinforced Concrete Project - Full scale tests are presently being conducted at Cornell, model and shaking table tests at Buffalo, and diaphragm tests at Lehigh. Tests are being performed on columns with short splices and widely spaced ties. Analytical work is being performed to compliment the testing. Preliminary information from this work should be available by the end of the summer. Chris said that it would be beneficial if these results could be incorporated into revisions to one or more of our

statements. It seems that the final results of these projects will not be available for inclusion into this work. Dick added that other tests are being planned for beams without continuous longitudinal reinforcement.

2) Semi-Rigid Connections in Steel Framed Structures - Three projects are presently underway to study this subject at the following Universities: SUNY Buffalo, South Carolina, and Minnesota. Testing is planned for this summer, with the reports to follow. Bob Hanson also is performing an analytical study on this subject at Michigan.

3) Expert Systems - Cornell and Lehigh are presently developing an Expert System for the seismic design of new buildings. Professor Bielak at Carnegie-Mellon is using ATC-14 as the basis for an Expert System for seismic evaluations. He will have developed the evaluation on one model building type (Unreinforced Masonry in regions of low seismicity) for the August Workshop. This project would like to incorporate the changes proposed by our study.

4) Seismic Evaluation of Buildings in New York City - Three groups are participating in this project. Professor Turkstra at PUNY and Professor Shinozuka at Princeton are performing damage assessment studies for classes of buildings and examining the societal impacts. Investigators at Cornell are examining typical building construction in New York City to determine testing requirements.

5) Ground Motion Studies - The NCEER Scientific Advisory Committee is presently requesting that Klaus Jacob develop an input ground motion that is typical for the Eastern United States. He may incorporate the Lawrence Livermore Code to estimate ground motion based on the distance and magnitude. NCEER is also attempting to purchase the results of the EPRI study. Paul Somerville of Woodward-Clyde Consultants may participate in this effort. The Lamont-Doherty group has developed a data base for all earthquakes world-wide, including spectra. This information should be accessible over phone lines within the next two months.

Chris stated that all of the information which would be applicable to this project could definitely improve the present document. Peter would like us to mention these other projects in a section of our report. We may want to identify statements and other portions of ATC-14 which could be affected by the results of these projects. Coordination of these results could be part of next years' work. Chris requested that Peter send a copy of the progress reports for each of the relevant projects for our use in preparing this portion of our report.

TASK #2 - Update A_a and A_v maps for the Eastern U.S. Based on Best Information Available

This task was addressed in the méeting which this project in conjunction with the February Conference in New York City. This meeting convened a number of seismologists who have been involved in recent research into this topic, such as EPRI and NCEER investigators. The meeting was well attended and generated a good deal of discussion which will hopefully lead to further NCEER sponsored research into this topic. The minutes recorded from this meeting will be included in the final report for this project. We will also include a short discussion of Klaus Jacob's work on developing a characteristic Eastern ground motion. Peter will send us a copy of the latest quarterly report on this project.

TASK #3- Develop a List of Historical Documents which Discusses Typical Construction Techniques

A large number of historical documents were identified and catalogued by Dick, Glen, and Jim. Two different formats were used to present the necessary information on these documents. It was decided that a combination of the two formats would be used to present this information in the final report. Dick will generate additional references from the architectural library at Cornell. He will not include the British texts in his search since the construction practices identified in these documents may not be applicable to typical United States construction. Glen and Jim will provide more information on the contents of the references they cited. Dick mentioned that the Strand Bookstore on Broadway in New York City may be the best source for obtaining these historical documents.

Chris stated that a possible future research project could examine these historical documents to evaluate critical and vulnerable details in relation to the recommendations of the ATC-14 document. We will recommend that this topic be the subject of a future research project.

TASK #4 - Develop of all Applicable Model Building Codes

Charles cautioned that identifying the date of inclusion of seismic provisions may be misleading since often these sections were included as an Appendix and were therefore not mandatory. Chris stated that ATC-21 lists the different codes and dates of adoption of seismic provisions. He added that it may be misleading to compare the formulas and/or the base shear formulas. Peter stated that the force level may not be a good method of comparison since the details are the critical not be a good method of comparison since the details are the critical elements. He will send us Shinozuka's study which compares wind and earthquake forces. Charles feels that it may be best not to compare these items since we cannot account for when the present provisions become obsolete. Chris also feels that our intent should only be to identify the dates of inclusion.He added that we may want to comment on the adequacy of the different provisions. Dick said that we should include a list of the codes used by the various federal government agencies, such as the Navy, VA, GSA, and the Tri-Services Manual, since these documents are often used even when not required by the local jurisdiction. Charles stated that a list of the states which have adopted specific model building codes and when the codes were mandated. Charles said that presently 32 states have adopted a model code. Charles will supply us with this information.

TASK #5 - Expand the List of Current Standards for Allowable Stresses

Dick began the discussion of this topic by stating that we should add the ASCE/ACI Committee 530 document titled "Proposed Building Code Requirements for Masonry Structures and Specifications for Masonry Structures". Charles added that important ICBO Research Standards should also be included. Dick stated that the design guide published by the Metal Building Manufacturers Association could also be useful. Charles stated that ACI Committee 349 report includes a discussion of standards for concrete anchorages in Appendix B. He also feels that the ACI Manual of Concrete Practice should be referenced, with specific sections noted where necessary. Glen said that the Steel Diaphragm Design Manual prepared by the Steel Deck Institute should be included. Dick added that the AISI Commentary on metal decking for walls and roofs is a six part series of reports with compliments the Manual. Charles added that Porter's work on metal deck with concrete fill may provide some useful information. This will not be included in this Section, since it is not an accepted standard, but we may want to list it with the other references. Chris added that we will include the addresses where these standards can be obtained.

TASK #6 - Develop a Recommended List of References and Standards from the Results of Tasks #4 and #5

The comments presented above will incorporated with the previously collected references and standards to complete these two recommended lists.

TASK #7 - Add Figures to the Text of ATC-14, as Appropriate

Degenkolb developed a set of proposed figures from the suggestions obtained at the first Project Meeting. A number of figures were proposed for the chapter on nonstructural elements. Dick stated that ACI Committee 551 has prepared a report on the state of the art of Tilt-Up Construction which is presently being reviewed by TAC. This report may include some figures and information which would be useful for our project. He will send us a copy of the draft, which cannot be quoted without the permission of ACI Committee 551.

TASK #8 - Prepare a Discussion of the Evaluation of Site Effects including Liquefaction, Landslide, and Faulting

Glen provided the group with the Massachusetts Code requirements for checking for liquefaction potential. He stated that these requirements, which relate to blow counts and depth of soil, were developed by the BSCE Geotechnical Section. He is not sure if they are applicable throughout the nation. Charles stated that there are curves to translate blow counts to Liquefaction Potential. He said that Dave Elton of Auburn and Wayne Clough of Virginia Tech are investigating this subject. Chris reminded the group that ATC-14 is addressed at life safety issues. He asked when a life safety issues occurs for liquefaction. Are there specific types of buildings that are susceptible to this hazard? He stated that differential settlement is the problem, so buildings that can "come apart", such as unreinforced masonry and precast concrete, are the most susceptible. He also added that tall, narrow structures (say H/D > 4) should also be flagged. Charles added that structures on spread footings and those that are marginally designed should also be of concern. Glen stated that to investigate the potential effects of a liquefaction problem an analysis which assumes a large local displacement or loss of bearing can be employed. Chris stated that five of the model building types are the most susceptible to this type of damage: W2, PC1, PC2, RM2, and URM. Charles cautioned that we may not know enough to limit this list to only five of the model building types. Dick said that we should state that these are the most susceptible. Jim added that we should provide the basic information on the characteristics that we feel are important in determining if a structure is susceptible to damage from liquefaction effects. Chris stated that we do not want to require any unnecessary analyses, since these procedures can easily become very expensive. Charles said that liquefaction is a larger problem in the East. Charles suggested we confirm our feelings on which building types are susceptible to liquefaction with Chris' partners at Degenkolb. He feels that we should present a general discussion on liquefaction from the results of Elton's work. Peter added that we should point out any further information which may be needed to address this subject.

On a similar note, Glen brought up the subject of tying together deep foundations, possibly to a level less than the 10% rule required for new construction. Jim informed this group that the original Degenkolb draft of ATC-14 included this requirement, but it was taken out by the Project Engineering Panel. Glen suggested a 2% tie requirement as a screen level to insure that a nominal connection between the pile caps is provided.

TASK #9 - Identify Eastern Earthquake Experience and Add to Appropriate Sections

Degenkolb collected a number of references which discuss Eastern earthquakes such as the 1886 Charleston, the New Madrid, and other smaller events. Charles and Glen will provide descriptions for addition to the examples of building performance which are presented in ATC-14. These examples will cover the performance of wood and unreinforced masonry structures in the Charleston and Cape Ann events. Dick mentioned that John Stevenson has completed his report for EPRI on the recent earthquake in Ohio.

TASK #10 - Rewrite Sections on Low Seismicity

Degenkolb collected all of the comments provided by the review panel and prepared the proposed revisions to the low seismicity sections of the evaluations. These proposed revisions were presented through use of the high seismicity checklists to reduce the volume of the submittal.

Chris began the discussion by questioning if all of these proposed additions to the low seismicity sections were only necessary for the evaluation of tall buildings. Glen thinks that all buildings should be included since most were not designed for seismic forces. Charles agreed with Glen. Chris noted that most of the statements which were proposed for addition related to tying the building together or the basic lateral strength. Chris believes that we should include the statements concerned with tying the building together. Charles added that the differences in the ABK methodology between URM buildings in regions of low and high seismicity is not that great, since the lower accelerations expected for these regions are balanced by longer durations. Chris agreed that duration is the major concern in that situation.

As the discussion of the specific additions began, it was decided that the statements that Glen developed for Task #11 on deterioration of structural elements should be addressed in conjunction with this task. These statements will be referred to as GB1, GB2, etc.

Page 89 - Add GB1 to Statement 5.5.3, and expand the procedure to provide guidance on reducing the capacity of the elements.

GB2 - This statement should be added with more discussion on how to discount areas which are deteriorated. Also, the statement should be revised to refer to wood and metal deck diaphragms.

GB3 - This statement should be added as Statement 26 in Chapter 11.

GB7 - This statement should be inserted in Section 11.2.10 as Statement 1f. Charles added that the difficulty occurs when the connection details do not allow movement. This concern should be added to the statement. Glen stated that he feels that Section 11.2.10 should not be separated into two sections (Exterior Cladding Elements and Connection Details). The panel agreed with this suggestion.

Page 154 - In Statement 7.1.5.3, we need to expand the description of how to consider the extent of the deterioration. Glen said that if the deterioration is local, an analysis can be performed to determine if the structure is adequate without including the deteriorated area. If the deterioration is extensive, material tests should be performed. This discussion should also be incorporated into Statements 8.1.5.3, 9.1.5.3, and 10.1.5.3.

GB4 and GB5 - In the "Procedure" portion of these statements, delete the phrase "due to ...", and add discussion similar to that prescribed for Statement 7.1.5.3.

GB6 - This statement addresses a cladding problem would be most appropriately located in Chapter 11.

The remaining discussion of this Task addressed the specific high seismicity statements which will be proposed for addition in the low seismicity lists. Each of the high seismicity checklists were marked with the proposed additions and revisions. The following paragraphs will refer to the checklist page in Appendix C which was the basis of the discussion.

Page C3 - The proposed revisions were acceptable to the group. Degenkolb suggested that low seismicity checklists be added to the document. The group agreed with this suggestion.

Page C4 - Statement 6.1.6.7 will not be included in the low seismicity checklists. Charles asked if we could use the NEHRP Commentary on putting setbacks on top of a building base. Chris said such an analysis is only necessary for cases where there is a drastic setback. He cited the old SEAOC provisions which recommended that 40% more mass be added in the analysis. Charles said that Engineers in the East may not be capable of performing such an analysis. Peter then questioned why pounding was only a concern when the floors do not meet. Glen stated that pounding could also be a problem in regions of low seismicity where the floors do not meet. Charles cited examples of large displacements which were reported after the 1886 Charleston Earthquake. He thinks the concern should also be added when the floors are at the same level. Dick said that if the problem were ever to occur, it would have happened during the 1985 Mexico City Earthquake. It was decided to add the pounding statement as it stands in ATC-14. Statements 6.1.6.11 and 6.1.6.15 will not be included in the low seismicity checklist.

in the low seismicity checklist.
Page C5 - Statement 6.2.6.1 will not be added since the revised version of Statement 6.2.5.3 is adequate to address this issue. Statement 6.2.6.10 will be added in its present form. Statement 6.2.6.12 will not be included in the low seismicity checklist. Dick stated that a discussion be added to each checklist concerning the use of Chapter 11. The "LS" issues in Chapter 11 should be addressed in all evaluations.

Page C6 - Chris questioned if a separate section should be included for this building type since almost all of the statements are to be included in regions of low seismicity. The group decided that to be consistent both sections and checklists should be provided.

Page C7 - Statement 6.4.6.10 will not be added to the low seismicity list since this concern will be addressed in the rapid shear stress check. The proposed minimum topping slab of 3 inch was accepted by the panel. Dick questioned if the intent of Statement 6.4.5.6 was to address a back-up wall rather than an exterior wall course. Chris agreed that a figure would help to explain the intent of this statement.

Page C8 - Statements 6.5.6.8-10 will be added to the low seismicity sections for buildings founded on soft soils (S_3 and S_4). Statement 6.5.6.12 will not be added since Glen's submittal on cladding will cover this subject.

Page C9 - Chris felt that the $8d_b$ may be too strict. Jim stated that this was taken from the ACI provisions for semi-ductile detailing. Charles confirmed that the SBC uses the same requirement. The group agreed to use this requirement. In Statement 7.1.6.9, it was decided to allow weak columns and strong beams if the strength is greater than $0.4R_w$. Degenkolb will check why this provision was not allowed for regions of high seismicity. Statement 7.1.6.20 will be modified to only require column ties through the joint for exterior and corner columns. This will agree with the requirements of the Massachusetts Code. In Statement 7.1.6.18, it was determined that d/2 should be the maximum beam tie spacing in order to agree with the ACI provisions.

Page C12 - Delete Statement 7.3.6.12 from the low seismicity section. We may want to delete this statement entirely since wood diaphragms probably would not be combined with cast-in-place concrete frames.

Page C13 - Chris questioned why Statement 8.1.6.10 was recommended for inclusion in the Low seismicity section. Glen said that Statement 8.1.5.7 will adequately address this issue.

Page C16 - Statement 9.1.6.1 is no longer needed due to the additions which Glen has proposed. The maximum wall anchor spacing requirement suggested for inclusion will be added to Statement 9.1.5.7.

Page C17 - Statement 9.2.5.7 will be revised to include the maximum wall anchor in a manner similar to page C16.

TASK #11 - Develop Statements for Deterioration Due to East Coast Environmental Effects

These statements were developed by Glen and discussed under Task #10.

TASK #12 - Develop Statements Related to Wharf Structures

Charles prepared a discussion of this topic and presented a statement to address this issue. He stated that the types of structures located in the flood plain typically include wood housing, condominiums with concrete walls, wharf facilities, and sometimes URM buildings over wharves, such as in Boston. The group discussed the statement proposed by Charles and made some minor editorial modifications.

TASK #13 - Develop Statements Related to Exterior Masonry Walls which are Braced by Vertical Load Carrying Frames of Steel or Concrete

Glen prepared this section with his Attachment H. He prepared a statement which checks that all masonry walls which are not part of the vertical or lateral force resisting systems are isolated from the structural frame. Chris asked why complete separation is necessary. Glen then discussed the problem of crushing the masonry and/or shearing the columns. Chris said that shearing the columns is only a concern for concrete frames. It was decided that this statement will not be added as part of the recommended revisions.

TASK #14 - Review Statements Related to Overturning in Concrete Shear Walls

Glen provided the group with some SGH reports which dealt with this subject. During the discussion of the Statements in Task #10, it was decided to leave the minimum H/D requirements presented in ATC-14.

TASK #15 - Identify the Most Critical Research Needs for the Different Building Types

Dick, Glen and Degenkolb all identified a number of research needs. Glen stated that cladding issues are a major problem, especially stone cladding. This topic is often overlooked as a research problem. Charles mentioned that Professor Kahn at Georgia Tech had been involved in this topic. Dick added that Professor Rihal at Cal Poly has also done a lot of work in this area. Chris suggested that we provide examples figures of poor details which could be the basis of future research. We should also check with Warner Howe since he has done a good deal of work on this topic.

TASK #16 - Review Statement 7.1.5.7

Glen still questions checking the shear stress rather than the bending stress, since he has found that bending always controls the capacity. Peter stated that if the drift is limited and the shear capacity is larger than the moment capacity, the system is adequate. Chris mentioned that some tests at the University of Texas demonstrated large cyclic displacement capacity without stability problems. Glen and Charles both stated that there are buildings that may be unstable even without lateral forces. Charles suggested that tall, slender buildings with light reinforcement could be in this group. He stated that over a certain height, such as four stories, the buildings may be checked. Dick agreed with the four floor limitation. Chris suggested that we add the drift check to the low seismicity section for buildings of three stories or more. More information on this topic from the upcoming tests at Cornell.

TASK #17 - Incorporate the Semi-Ductile Requirements for Areas of Low Seismicity

Degenkolb revised the requirements provided for ductile detailing of concrete structures to correspond with the semi-ductile requirements of ACI and the Mass. Code. These issues were addressed and resolved in the discussion of Task #10.

TASK #18 - Review the Appropriateness of the .005H Drift Limit for All Building Types

Peter prepared a discussion of this topic which presented some of the various drift limits proposed by various researchers for different structural and nonstructural elements.

Peter started the discussion of this topic by stating that drift is a good index of damage, but the problem is how to accurately calculate the displacements. His information indicates that the .005H limit is not good for all systems, and is much too small for frame systems. He feels that the limit we use much be related to how the displacements are calculated. One possibility that Peter mentioned was that we could analyze the deflections to drift instability using guidelines employed for tall buildings.

Charles stated that it may not be appropriate to use a requirement which is less stringent than accepted code levels; our level should be tied to some standard. Chris said that the present ATC-14 document attempted to follow the accepted drift limit requirements, and that changing these would be moving away that position. Peter agreed that what we have now is acceptable, and noted that there will be more research on this topic in the future.

TASK #19 - Develop Statements on Tile Arch Floors

Glen prepared a statement on this subject which is intended to alert the evaluator to the possible danger of clay-tile arch floors. Glen mentioned that the collapse of a tile floor in the Skopje Earthquake was not caused by falling tiles. Glen is not concerned with solid brick arches, but rather with clay-tile arches. He stated that the clay-tiles could shatter and fall from the shear racking imposed on the tile by the displacement of the concrete floor. Glen stated that the procedure to analyze this condition consists of analyzing the displacement of the concrete topping slab due to diaphragm shear and imposing that displacement on the clay tiles. The diagonal tension stress in the clay tiles are then analyzed to insure that cracking of the tile is avoided. Charles added that the compressive stresses induced by the arch action may help to the hold the system together. Charles and Dick agreed that the statement and procedure are reasonable. Dick cautioned that the concrete topping slab would probably be badly cracked due to shrinkage and other effects. Chris asked how the allowable stresses would be determined. Glen said that some of the historical documents could be useful in this regard. He added that the Structural Clay Products Institute (SCPI) will provide some information for modern products. Dick added that older references would probably only give compressive stress information. Glen will try to get more information on this topic and change the procedure to be more prescriptive, providing discussion on calculating the diaphragm shear stresses, etc. An allowable shear stress in the clay-tile will be recommended by Glen. A statement will be added to the "Concern" portion of the statement to the effect that solid brick arches are not of concern.

TASK #20 - Develop Statements for Brick and Rubble Stone Footings

Charles prepared a discussion of this topic and Glen submitted some information on foundation construction from Kidder's book on building construction. From this information, Degenkolb developed a statement on this topic.

Charles stated that we need to look at a larger group of foundation types that also have limited lateral force resisting capacity, such as timber grillages or platforms, and steel platforms. He added that pile foundations and spread footings without ties are also suspect.

Chris suggested that we add other types of foundations to the statement and figures (possibly from Kidder) to clarify the intent. This statement should be added to all building types. He also suggested that a discussion of possible foundation exploration work should be added to Chapter 4. He added that we should add a sentence to alert the concern regarding pile foundations. He also recommended that the

TASK #21 - Expand the Nonstructural Descriptions to Provide More Information to the User

Degenkolb prepared a substantial modification to the contents of Chapter 11, which included expanded descriptions and a number of proposed figures.

Charles initiated a discussion on the potential hazards of elevator machinery. He noted three areas of concern: 1) attachment and stability of the guide rail, 2) restraint of the cable housing from jumping off the drum, and 3) anchorage of the counterweight. Charles proposed that he prepare two new statements to address the first two concerns, both of which are life safety issues.

Charles then asked about large cabinets and contents such as shelving, books, chemicals, water heaters, etc. A section on contents will be added to Chapter 11, including life safety issues where appropriate. Glen said that Statement 11.2.1 on page 254 should be expanded to require that all partitions be braced at the top.

TASK #22 - Expand Chapter 11 to Include All Wall Systems Used in the United States

Glen prepared a large new section on cladding, glazing and veneer, which eight different forms of curtain wall systems.

A discussion of the appropriate procedure to perform a qualitative field test of the mortar joints resulted in the decision to change the wording from "be scraped out of them by fingernail" to "be easily scraped from the joints".

Chris suggested that the statements such as #3 in Section 1 of Glen's submittal should only be applicable for moment frames. This modification will be added to these statements. In Section 2, it would be useful to provide a figure in conjunction with statement #2 on through wall flashing. Also, add the phrase "in the vicinity of" to this statement. Dick suggested that statements 2, 4, 8 and 9 should be listed consecutively. Change the 6 to 4 in statement 11 of Section 2. The word "Panel" should be added to the end of the title of Sections 4, 5, and 7. In Section 5, delete statement 5, and add "curtain" to locations where the word "wall" occurs. Glen will provide representative figures which would be beneficial to this section.

V. **SUMMARY OF THE MEETING AND SCHEDULE** - Chris listed the tasks to the members of the panel which were agreed to during the meeting. Charles will prepare a summary discussion of the differences between the East and West which will be reviewed and added to (if necessary) by Glen and Warner.

Proposed Schedule for the Remainder of the Project

1. Completion of all remaining tasks for submittal to Degenkolb by July 18.

2. Draft Report to be completed by August 5.

3. Panel Comments on the Draft Report by August 15.

4. Submission to NCEER by August 19.

Proposed Format of the Report

A total of eight sections are envisioned for the final report on the results of this project. The format of the report will follow NCEER Standards as much as possible, but will retain the ATC-14 style where new statements are proposed. The following eight sections are planned for the report:

1. Introduction

2. Discussion of Liaison with other NCEER Projects, including description of possible statements to be affected by these projects.

3. Seismicity Issues, based on the results of the February meeting and the progress reports.

4. Discussion of the Differences in Practice

5. The Lists of Historical Documents, Reference Standards, etc.

6. Rewrite of the Sections on Areas of Low Seismicity

7. Rewrite of Chapter 11

8. List of Errata

Discussion of Possible Continuation of This Work

Peter asked how the work begun by this project could be continued. Two major branches related to this project were discussed. The first was the technical continuation of the project, which could include the following:

1. Research applications to issues identified in ATC-14.

2. Trial evaluations of representative buildings from all around the country.

3. Relating the model buildings to the historical design documents.

4. Sponsor a review of the document by local ASCE Sections to encourage a broad based involvement of the engineering community.

The second area of continued work would by the technology transfer of the information developed in ATC-14 and this project to the design engineers across the country. This would consist of seminars, workshops and other meetings which would increase the awareness of document. Dick mentioned that a follow-up to the New York City Conference is a possibility. Charles would like to plan an ATC-14 seminar for next February.

08/10/88
d/m-r

APPENDIX B

PRELIMINARY PROCEDURE FOR THE

EVALUATION OF LIQUEFACTION POTENTIAL

Preliminary Procedure for the Evaluation of Liquefaction Potential

The seismic hazard of soil liquefaction is discussed in Chapter 3. This Chapter recommends that the liquefaction potential at a site be assessed and, if found to be positive, that the technical problem be referred to a qualified geotechnical engineer for resolution. As related in Section 3.2.6, this Appendix presents the basic procedure for evaluating liquefaction potential. The procedure is as described by Clough (1988) and Elton (1988) and is based on developments of others (Seed and Idriss, 1982; Seed and De Alba, 1986; Marcuson and Bieganousky, 1977).

Step 1. Calculate cyclic shear stress induced in the soil deposit at various depths by earthquake ground motion and convert the irregular stress histories to equivalent numbers of uniform stress cycles. In this manner, account is taken of the intensity of ground shaking, the duration of shaking, and the variation of induced shear stress with depth. A plot of the induced equivalent uniform shear stress level as a function of depth is produced like that shown in the dashed curve (Curve A) in Figure B.1.

Step 2. Calculate the cyclic shear stress that would have to be developed at various depths in order to cause liquefaction to occur in the same number of stress cycles as that determined in Step 1 to be representative of the particular earthquake under consideration. In this manner, consideration is made of the soil type, the in-place conditions, the seismic and geologic histories of the deposit, and the initial effective stress conditions. The computed stress required to cause liquefaction can then be plotted as a function of depth as shown in the solid curve (Curve B) in Figure B.1.

Step 3. Determine whether any zone exists within the deposit that liquefaction can be expected to occur by comparing the shear stress induced by the earthquake with that required to cause liquefaction (induced stress exceeds that required to liquefy).

A simplified approach developed by Seed (1979) is used to calculate the average earthquake-induced shear stress as required in Step 1 above:

$$\tau = 0.65\, a_{max}\ \sigma_o\, r_d \qquad (B.1)$$

where:

τ = cyclic shear stress applied to ground

σ_o = total overburden stress at the depth of concern

r_d = reduction factor for soil flexibility varying from 1 at the surface to approximately 0.9 at a depth of 30 feet (10 m)

a_{max} = maximum peak ground acceleration in g's expected at the site under consideration

The maximum acceleration (EPA) is computed considering the likely size of earthquake, the attenuation effects that might occur over the distance between the site and the earthquake epicenter, and any potential magnification of the earthquake waves during their propagation through the near surface materials (Clough, 1988).

The reduction factor can be calculated with the following equation (Iwasaki, 1981):

$$r_d = 1 - 0.015d \qquad (B.2)$$

Equation B.1 is usually normalized by dividing both sides by the effective vertical overburden stress σ_o at the depth of concern. The result is the cyclic stress ratio (CSR), as indicated in Equation B.3:

$$CSR = \frac{\tau}{\sigma_o} \qquad (B.3)$$

According to Step 2, the cyclic strength is calculated next using Standard Penetration boring logs common to production engineering. The cyclic strength is determined in a normalized form as the ratio of cyclic strength to effective overburden pressure. This ratio is termed the critical cyclic stress ratio (CCSR). As implied in Step 3 of the procedure, liquefaction is likely to occur if the cyclic stress ratio (CSR) exceeds the critical cyclic stress ratio (CCSR). The critical cyclic stress ratio required to cause liquefaction (Step 2) can be evaluated from empirical relationships as developed by Seed and De Alba (1986) for clean sands and silty sands. Shown in Figure B.2, the curves plot the cyclic stress ratio versus the normalized standard penetration resistance of the soil at sites that experienced earthquake shaking. The penetration resistance is specified in terms of their respective corrected SPT N-values that will be explained below.

Separate sites that liquefied (left of curve) and sites that did not liquefy (right of curve) are identified. The illustrated curves were developed from liquefaction data from all over the world for earthquakes of surface wave magnitude $M_s = 7.5$ and for different fines content.

As discussed by Elton and Hadj-Hamou (1988), the fines content of a cohesionless soil (percentage of particles passing through a no. 200 sieve) influences the resistance to liquefaction. Increasing fines content tends to reduce the build-up of pore pressures that lead to liquefaction during the earthquake. The magnitude of the earthquake affects the number of cycles of loading felt by the soil. The larger earthquakes produce more cycles of loading felt by the soil. The larger earthquakes produce more cycles of loading and, hence, more readily liquefy the soil. The cyclic stress ratio for other earthquake magnitudes is obtained by using the correction factor from Table B.1 to the cyclic stress ratio for the $M_s = 7.5$ (After Seed and De Alba, 1986).

TABLE B.1

Correction Factors for Influence of Earthquake Magnitude on Liquefaction Resistance

Richter Magnitude	Correction Factor	Number of Representative Cycles at 0.65 a_{max}
5.25	1.50	2 - 3
6.00	1.32	5 - 6
6.75	1.13	10
7.50	1.00	15
8.50	0.89	26

In addition, the normalized SPT N-values in the empirical relationship are corrected for overburden pressure (Marcuson and Bieganousky, 1977) and for the energy ratio of the hammer used in the investigation (Seed and De Alba, 1986) as discussed by Elton and Hadj-Hamou 1988). The two corrections are applied and the corrected SPT values obtained using the following equation:

$$N_c = N \times ER \times C_n \qquad \text{(B.4)}$$

where:

N_c = corrected N-value

ER = correction factor for energy ratio

C_n = correction factor for overburden pressure

For a donut hammer, ER is equal to 60/45 = 0.75. The value C_n is taken from Figure B.3.

Sometimes it is useful to estimate the unit weight of the soil from the soil type and the penetration values provided by Bowles (1982) provided in Table B.2.

TABLE B.2

Relationship Between SPT Values and Density (after Bowles, 1982)

Cohesionless Soils		Cohesive Soils	
N-Value	Unit Weight (pcf)	N-Value	Unit Weight (pcf)
5 - 10	80 - 110	2	100 - 120
8 - 15	90 - 130	4 - 8	110 - 130
10 - 40	130 - 140	16 - 32	120 - 140
20 - 70	140 - 150		

The N-value in Equation B.4 is determined using the SPT procedure prescribed by the American Society of Testing and Materials (ASTM) Standard D1586-84 (1986). A 140-pound (63.5 kg) hammer is dropped 30 inches (76 cm) onto an anvil attached to the drill rods. The hammer is typically held by a jute rope, which is wrapped twice around a motorized cathead, which raises the hammer. A 2 inch (5.1 cm) outside diameter, 1.4 inch (3.5 cm) inside diameter, split spoon sampler is attached to the drill rods, and driven 18 inches (46 cm) into the bottom of a borehole. The number of blows (N-values) was recorded from 6 to 18 inches (15 to 46 cm) during the driving of the tool.

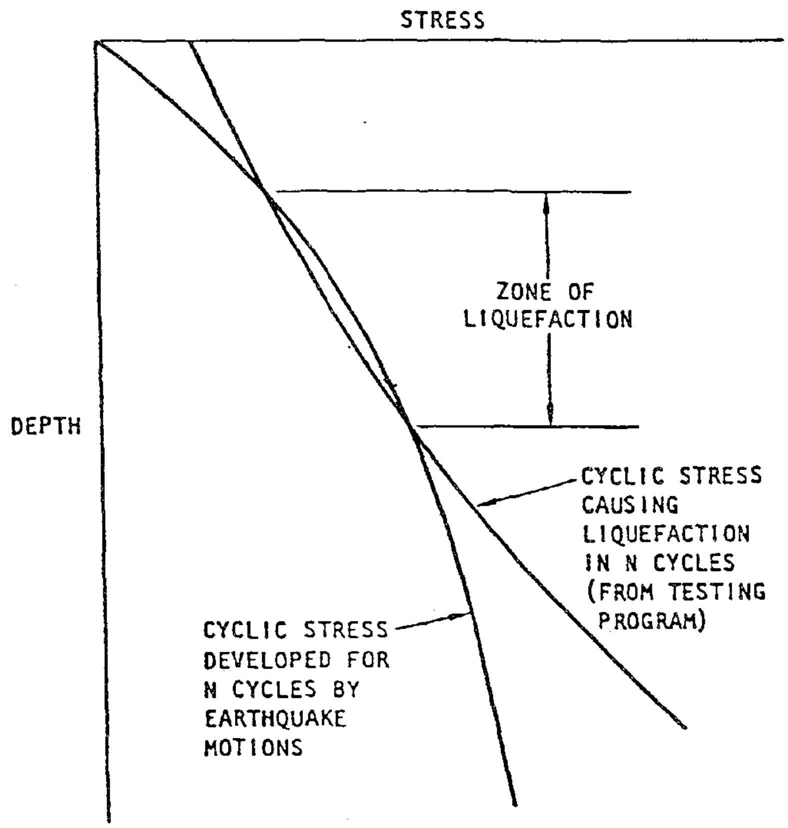
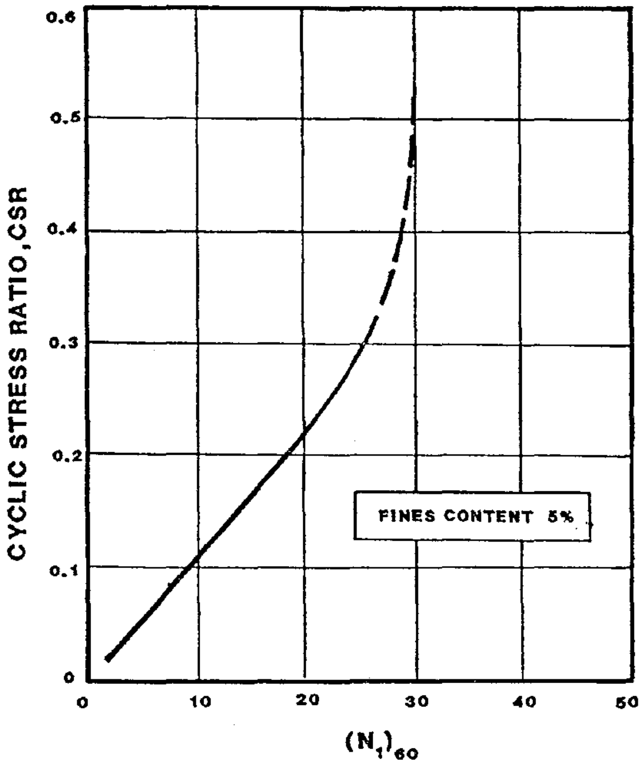


Figure B.1 - Method of Evaluating Liquefaction Potential
(Seed and Idriss, 1982)



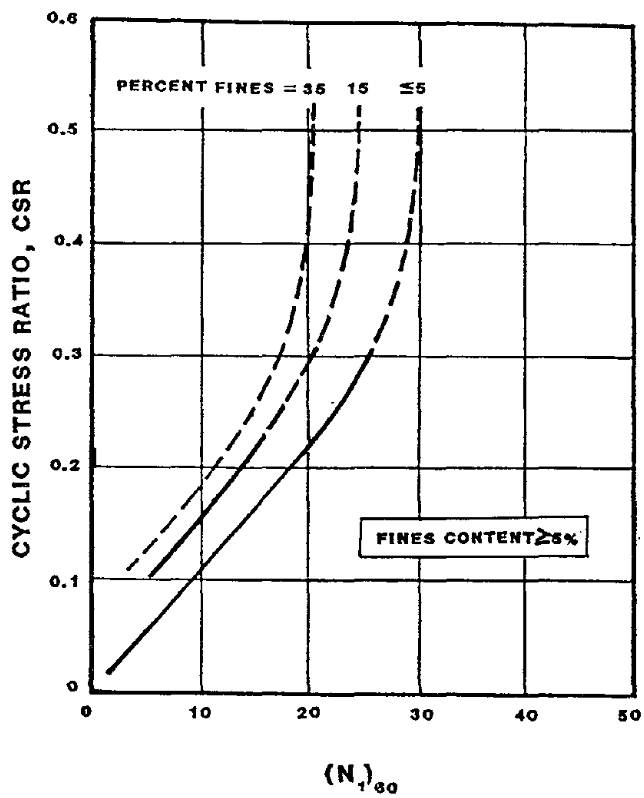


Figure B.2 - Empirical Relationship Between Shear Stress Ratio and SPT (Seed and De Alba, 1986)

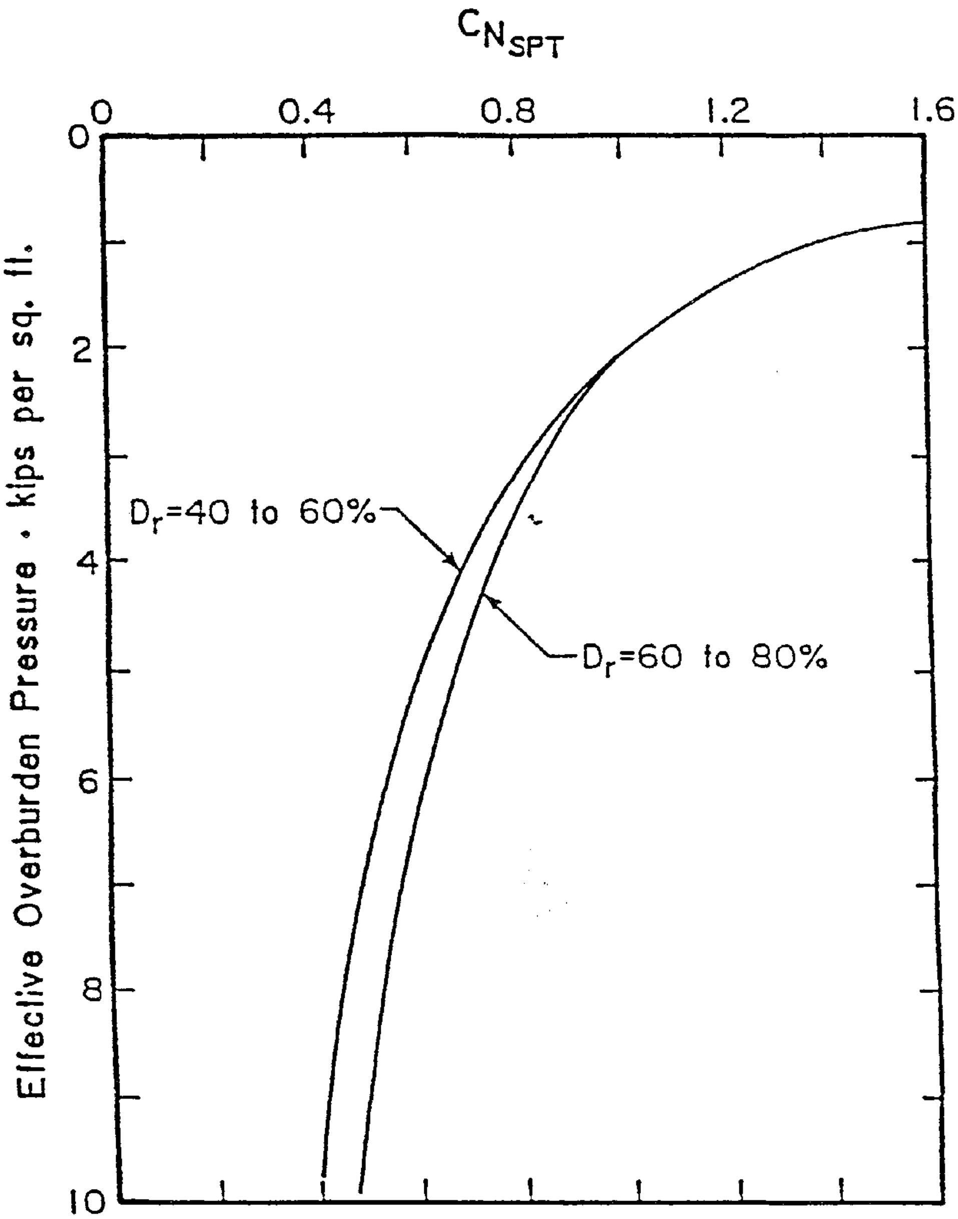


Figure B.3 - Overburden Correction Factor for N-Value (Marcuson and Bieganousky)

APPENDIX C

CHECKLISTS FOR BUILDINGS

IN REGIONS OF LOW SEISMICITY

Checklist 1. Wood Buildings*
(Low Seismicity Regions)
Type 1 - Dwellings
Type 2 - Commercial or Industrial

True/
False   Comments

MATERIALS

____ 5.5.1 No signs of decay, sagging, splitting of wood or deterioration of metal accessories.

____ 5.5.2 No substantial leakage damage to roof deck.

STRUCTURAL ELEMENTS

____ 5.5.3 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 5.5.4 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 5.5.5 Large openings (re garage doors) are braced or tied-in.

____ 5.5.6 Walls are bolted to sill at 6 feet or less spacing.

FOUNDATIONS

____ 5.5.7 Posts are positively connected to foundation.

____ 5.5.8 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

____ 5.5.9 The foundation is not composed of unreinforced masonry or stone rubble.

____ 5.5.10 There is no foundation or superstructure damage due to heaving soil.

True/
<u>False</u> | <u>Comments</u>

<u>NON-STRUCTURAL ELEMENTS</u>

____ 5.5.11 Exterior cladding and veneer are well anchored.

____ 5.5.12 Reinforced masonry chimneys are tied into all diaphragms.

*See Section 6.1 (ATC-14 - Chapter 5, Section 5.5), for detailed discussion on each of these checklist issues.

Checklist 2. Steel Moment Frame Buildings*
(Low Seismicity Regions)

True/
False | | Comments

MATERIALS

____ 6.1.5.1 No signs of significant deterioration in vertical or lateral force resisting system.

____ 6.1.5.2 No substantial leakage damage to roof deck.

____ 6.1.5.3 No damage to masonry and/or concrete elements due to freeze/thaw action.

____ 6.1.5.4 No damage to concrete surfaces due to chloride-laden concrete.

STRUCTURAL ELEMENTS

____ 6.1.5.5 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 6.1.5.6 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 6.1.5.7 No vertical strength discontinuities.

____ 6.1.5.8 No torsion.

____ 6.1.5.9 No vertical mass or geometric irregularities.

____ 6.1.5.10 No pounding of adjacent structures.

____ 6.1.5.11 Chords around diaphragm openings greater than 50 percent of the width.

____ 6.1.5.12 Large tensile capacity at re-entrant corners or other plan irregularities.

True/
False | Comments

FOUNDATIONS

____	6.1.5.13	Columns are well anchored to foundation.
____	6.1.5.14	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.
____	6.1.5.15	The foundation is not composed of unreinforced masonry or stone rubble.
____	6.1.5.16	There is no foundation or superstructure damage due to heaving soil.

NON-STRUCTURAL ELEMENTS

____	6.1.5.17	Cornices, parapets, and other appendages are reinforced and anchored to the structure.
____	6.1.5.18	Exterior cladding and veneer are well anchored.

*See Section 6.2 (ATC-14 - Chapter 6, Section 6.1.5), for detailed discussion on each of these checklist issues.

Checklist 3. **<u>Braced Steel Frame Buildings</u>***
(Low Seismicity Regions)

True/
<u>False</u> <u>Comments</u>

<u>MATERIALS</u>

____ 6.2.5.1 No signs of significant deterioration in vertical or lateral force resisting system.

____ 6.2.5.2 No substantial leakage damage to roof deck.

____ 6.2.5.3 No damage to masonry and/or concrete elements due to freeze/thaw action.

____ 6.2.5.4 No damage to concrete surfaces due to chloride-laden concrete.

<u>STRUCTURAL ELEMENTS</u>

____ 6.2.5.5 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 6.2.5.6 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 6.2.5.7 No vertical strength discontinuities.

____ 6.2.5.8 No torsion.

____ 6.2.5.9 No vertical mass or geometric irregularities.

____ 6.2.5.10 Braced connections develop yield capacity of the diagonals.

____ 6.2.5.11 Chords around diaphragm openings greater than 50 percent of the width.

____ 6.2.5.12 Large tensile capacity at re-entrant corners or other plan irregularities.

True/
False		Comments

True/False			Comments
	FOUNDATIONS		
____	6.2.5.13	Columns are well anchored to foundation.	
____	6.2.5.14	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.	
____	6.2.5.15	The foundation is not composed of unreinforced masonry or stone rubble.	
____	6.2.5.16	There is no foundation or superstructure damage due to heaving soil.	
	NON-STRUCTURAL ELEMENTS		
____	6.2.5.17	Cornices, parapets, and other appendages are reinforced and anchored to the structure.	
____	6.2.5.18	Exterior cladding and veneer are well anchored.	

*See Section 6.3 (ATC-14 - Chapter 6, Section 6.2.5), for detailed discussion on each of these checklist issues.

Checklist 4. Light Steel Moment Frame Buildings With Longitudinal Tension Only Bracing*

(Low Seismicity Regions)

True/
False   Comments

MATERIALS

____ 6.3.5.1 No signs of significant deterioration in vertical or lateral force resisting system.

____ 6.3.5.2 No substantial leakage damage to roof deck.

____ 6.3.5.3 No damage to masonry and/or concrete elements due to freeze/thaw action.

____ 6.3.5.4 No damage to concrete surfaces due to chloride-laden concrete.

STRUCTURAL ELEMENTS

____ 6.3.5.5 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 6.3.5.6 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 6.3.5.7 No vertical strength discontinuities.

____ 6.3.5.8 No torsion.

____ 6.3.5.9 Light metal roof panels connected to roof framing at 12 inch maximum.

____ 6.3.5.10 Wall panels are connected to framing.

____ 6.3.5.11 Chords around diaphragm openings greater than 50 percent of the width.

____ 6.3.5.12 Large tensile capacity at re-entrant corners or other plan irregularities.

True/
False | | Comments

FOUNDATIONS

____ 6.3.5.13 Columns are well anchored to foundation.

____ 6.3.5.14 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

____ 6.3.5.15 The foundation is not composed of unreinforced masonry or stone rubble.

____ 6.3.5.16 There is no foundation or superstructure damage due to heaving soil.

NON-STRUCTURAL ELEMENTS

____ 6.3.5.17 Cornices, parapets, and other appendages are reinforced and anchored to the structure.

____ 6.3.5.18 Exterior cladding and veneer are well anchored.

*See Section 6.4 (ATC-14 - Chapter 6, Section 6.3.5), for detailed discussion on each of these checklist issues.

Checklist 5. Steel Frame Buildings With Cast-In-Place Concrete Walls*
(Low Seismicity Regions)

True/ False			Comments
True	RAPID EVALUATION OF SHEAR STRESS IN CONCRETE WALLS REQUIRED		
	MATERIALS		
____	6.4.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
____	6.4.5.2	No substantial leakage damage to roof deck.	
____	6.4.5.3	No damage to masonry and/or concrete elements due to freeze/thaw action.	
____	6.4.5.4	No damage to concrete surfaces due to chloride-laden concrete.	
	STRUCTURAL ELEMENTS		
____	6.4.5.5	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
____	6.4.5.6	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
____	6.4.5.7	No vertical strength discontinuities.	
____	6.4.5.8	Wall reinforcing greater than 0.0025 Ag each way with a maximum spacing of 18 inches.	
____	6.4.5.9	Metal deck has topping slab with minimum thickness of 3 inches.	
____	6.4.5.10	No torsion.	
____	6.4.5.11	No vertical mass or geometric irregularities.	
____	6.4.5.12	Reinforcing in each diaphragm to transfer load to walls.	

True/ False			Comments
____	6.4.5.13	Walls are continuous to foundation.	
____	6.4.5.14	Positive connection between walls and steel frame members.	
____	6.4.5.15	Chords around diaphragm openings greater than 50 percent at the width.	
____	6.4.5.16	Large tensile capacity at re-entrant corners or other plan irregularities.	
____	6.4.5.17	Diaphragm openings at walls are less than 25 percent of the length.	
____	6.4.5.18	Special wall reinforcement placed around all openings.	
____	6.4.5.19	Coupling beam stirrups spaced at $8d_b$ or less and anchored into each core with hooks of 135 degrees or more.	
	FOUNDATIONS		
____	6.4.5.20	Vertical wall reinforcing is doweled into the foundation.	
____	6.4.5.21	Frame columns are well anchored to the foundation.	
____	6.4.5.22	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.	
____	6.4.5.23	The foundation is not composed of unreinforced masonry or stone rubble.	
____	6.4.5.24	There is no foundation or superstructure damage due to heaving soil.	
____	6.4.5.25	Buildings taller than six stories are not founded on soils subject to liquefaction.	

True/
False | | Comments

NON-STRUCTURAL ELEMENTS

____ 6.4.5.26 Cornices, parapets, and other appendages are reinforced and anchored to the structure.

____ 6.4.5.27 Exterior cladding and veneer are well anchored.

*See Section 6.5 (ATC-14 - Chapter 6, Section 6.4.5), for detailed discussion on each of these checklist issues.

Checklist 6. **<u>Steel Frame Buildings With</u>**
<u>Infilled Walls of Unreinforced Masonry</u>*
(Low Seismicity Regions)

True/
False | Comments

MATERIALS

____ 6.5.5.1 No signs of significant deterioration in vertical or lateral force resisting system.

____ 6.5.5.2 Mortar quality - can't scrape with metal tool, and no large areas of eroded mortar.

____ 6.5.5.3 No substantial leakage damage to roof deck.

____ 6.5.5.4 No damage to masonry and/or concrete elements due to freeze/thaw action.

____ 6.5.5.5 No damage to concrete surfaces due to chloride-laden concrete.

STRUCTURAL ELEMENTS

____ 6.5.5.6 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 6.5.5.7 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 6.5.5.8 No vertical strength discontinuities.

____ 6.5.5.9 Exterior concrete or masonry walls are anchored to each of the diaphragm levels.

____ 6.5.5.10 Steel frames form a complete vertical system.

____ 6.5.5.11 No torsion.

____ 6.5.5.12 Infilled walls are continuous to base of building.

True/
False

Comments

____ 6.5.5.13 For buildings founded on soft soils (S_3 and S_4), height/thickness (h/t) of walls in one-story buildings are less than 14.

____ 6.5.5.14 For buildings founded on soft soils (S_3 and S_4), height/thickness (h/t) of top story walls in multi-story buildings are less than 9.

____ 6.5.5.15 For buildings founded on soft soils (S_3 and S_4), height/thickness (h/t) of walls in other stories in multi-story buildings are less than 20.

____ 6.5.5.16 Infilled panels are anchored to the steel frames around the entire perimeter.

____ 6.5.5.17 Chords around diaphragm openings greater than 50 percent at the width.

____ 6.5.5.18 No clay-tile arch floors are present.

FOUNDATIONS

____ 6.5.5.19 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

____ 6.5.5.20 The foundation is not composed of unreinforced masonry or stone rubble.

____ 6.5.5.21 There is no foundation or superstructure damage due to heaving soil.

____ 6.5.5.22 Buildings taller than six stories are not founded on soils subject to liquefaction.

NON-STRUCTURAL ELEMENTS

____ 6.5.5.23 Cornices, parapets, and other appendages are reinforced and anchored to the structure.

____ 6.5.5.24 Exterior cladding and veneer are well anchored.

*See Section 6.6 (ATC-14 - Chapter 6, Section 6.5.5), for detailed discussion on each of these checklist issues.

Checklist 7. Concrete Moment Frame Buildings*
(Low Seismicity Regions)

True/ False			Comments
True	RAPID EVALUATION OF REINFORCED COLUMNS REQUIRED		
True	Rapid Evaluation of Story Drift		
	MATERIALS		
____	7.1.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
____	7.1.5.2	No damage to masonry and/or concrete elements due to freeze/thaw action.	
____	7.1.5.3	No damage to concrete surfaces due to chloride-laden concrete.	
	STRUCTURAL ELEMENTS		
____	7.1.5.4	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
____	7.1.5.5	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
____	7.1.5.6	No vertical strength discontinuities.	
____	7.1.5.7	The shear capacity of the frame is greater than the moment capacity.	
____	7.1.5.8	No infills of concrete or masonry.	
____	7.1.5.9	No torsion.	
____	7.1.5.10	No vertical mass or geometric irregularities.	
____	7.1.5.11	Frames are continuous to the base.	
____	7.1.5.12	Strong columns - weak beams.	

True/ False			Comments
____	7.1.5.13	Metal deck has topping slab with a minimum thickness of 3 inches.	
____	7.1.5.14	No pounding of adjacent structures.	
____	7.1.5.15	Column ties at maximum of d over entire length, and at maximum of 8 d_b or d/2 at hinge locations.	
____	7.1.5.16	Column lap splice lengths are greater than 30 d_b.	
____	7.1.5.17	The positive moment strength at the face of the joint is greater than 1/3 of the negative moment strength. At least 20% of the steel is continuous.	
____	7.1.5.18	Beam stirrups at maximum of d/2 over entire length, and at maximum of 8 d_b or d/4 at hinge locations.	
____	7.1.5.19	Bent-up longitudinal steel is not used for shear reinforcement.	
____	7.1.5.20	Column ties extend through all joints.	
____	7.1.5.21	Large tensile capacity at re-entrant corners or other plan irregularities.	
____	7.1.5.22	Chords around diaphragm openings greater than 50 percent at the width.	
	FOUNDATIONS		
____	7.1.5.22	All column steel is doweled into the foundation.	
____	7.1.5.23	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.	
____	7.1.5.24	The foundation is not composed of unreinforced masonry or stone rubble.	
____	7.1.5.25	There is no foundation or superstructure damage due to heaving soil.	

True/ False			Comments
	NON-STRUCTURAL ELEMENTS		
____	7.1.5.26	Cornices, parapets, and other appendages are reinforced and anchored to the structure.	
____	7.1.5.27	Exterior cladding and veneer are well anchored.	

*See Section 6.7 (ATC-14 - Chapter 7, Section 7.1.5), for detailed discussion on each of these checklist issues.

Checklist 8. Concrete Shear Wall Buildings*
(Low Seismicity Regions)

True/ False			Comments
True	RAPID EVALUATION OF SHEAR STRESS IN CONCRETE WALLS REQUIRED		
	MATERIALS		
____	7.2.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
____	7.2.5.2	No evidence of corrosion of spalling at post-tensioning or end fittings.	
____	7.2.5.3	No damage to masonry and/or concrete elements due to freeze/thaw action.	
____	7.2.5.4	No damage to concrete surfaces due to chloride-laden concrete.	
	STRUCTURAL ELEMENTS		
____	7.2.5.5	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
____	7.2.5.6	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
____	7.2.5.7	No vertical strength discontinuities.	
____	7.2.5.8	Wall reinforcing greater than 0.025 A_g each way with a maximum spacing of 18 inches.	
____	7.2.5.9	Metal deck has topping slab with a minimum thickness of 3 inches.	
____	7.2.5.10	No torsion.	
____	7.2.5.11	No vertical mass or geometric irregularities.	
____	7.2.5.12	Walls are continuous to foundations.	
____	7.2.5.13	Reinforcing in each diaphragm to transfer loads to walls.	

True/ False			Comments
____	7.2.5.14	Chords around diaphragm openings greater than 50 percent of the width.	
____	7.2.5.15	Large tensile capacity at re-entrant corners or other plan irregularities.	
____	7.2.5.16	Diaphragm openings at walls are less than 25 percent of the length.	
____	7.2.5.17	Special reinforcement around all wall openings.	
	FOUNDATIONS		
____	7.2.5.18	Vertical wall reinforcing is doweled into the foundation.	
____	7.2.5.19	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.	
____	7.2.5.20	The foundation is not composed of unreinforced masonry or stone rubble.	
____	7.2.5.21	There is no foundation or superstructure damage due to heaving soil.	
____	7.2.5.22	Buildings taller than six stories are not founded on soils subject to liquefaction.	
	NON-STRUCTURAL ELEMENTS		
____	7.2.5.23	Cornices, parapets, and other appendages are reinforced and anchored to the structure.	
____	7.2.5.24	Exterior cladding and veneer are well anchored.	

*See Section 6.8 (ATC-14 - Chapter 7, Section 7.2.5), for detailed discussion on each of these checklist issues.

Checklist 9. Concrete Frame Buildings With Infilled Walls of Unreinforced Masonry*
(Low Seismicity Regions)

True/ False			Comments
	MATERIALS		
____	7.3.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
____	7.3.5.2	Mortar quality - can't scrape with metal tool, and no large areas of eroded mortar.	
____	7.3.5.3	No damage to masonry and/or concrete elements due to freeze/thaw action.	
____	7.3.5.4	No damage to concrete surfaces due to chloride-laden concrete.	
	STRUCTURAL ELEMENTS		
____	7.3.5.5	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
____	7.3.5.6	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
____	7.3.5.7	No vertical strength discontinuities.	
____	7.3.5.8	The exterior concrete or masonry walls are anchored to each of the diaphragm levels.	
____	7.3.5.9	The infilled walls are continuous to the soffits of the frame beams.	
____	7.3.5.10	Concrete frames form a complete vertical system.	
____	7.3.5.11	No torsion.	
____	7.3.5.12	Infilled walls are continuous to the base of the building.	

True/
False

Comments

____ 7.3.5.13 For buildings founded on soft soils (S_3 and S_4), height/thickness (h/t) of walls in one-story buildings are less than 14.

____ 7.3.5.14 For buildings founded on soft soils (S_3 and S_4), height/thickness (h/t) of top story walls in multi-story buildings are less than 9.

____ 7.3.5.15 For buildings founded on soft soils (S_3 and S_4), height/thickness (h/t) of walls in other stories in multi-story buildings are less than 20.

____ 7.3.5.16 Infilled walls are not of cavity construction.

____ 7.3.5.17 Infilled panels are anchored to the concrete frames around the entire perimeter.

____ 7.3.5.18 Chords around diaphragm openings greater than 50 percent of the width.

____ 7.3.5.19 No clay-tile arch floors are present.

FOUNDATIONS

____ 7.3.5.20 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

____ 7.3.5.21 The foundation is not composed of unreinforced masonry or stone rubble.

____ 7.3.5.22 There is no foundation or superstructure damage due to heaving soil.

____ 7.3.5.23 Buildings taller than six stories are not founded on soils subject to liquefaction.

NON-STRUCTURAL ELEMENTS

____ 7.3.5.24 Cornices, parapets, and other appendages are reinforced and anchored to the structure.

____ 7.3.5.25 Exterior cladding and veneer are well anchored.

*See Section 6.9 (ATC-14 - Chapter 7, Section 7.3.5), for detailed discussion on each of these checklist issues.

Checklist 10. Tilt-Up Buildings*
(Low Seismicity Regions)

True/ False			Comments
	MATERIALS		
____	8.1.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
____	8.1.5.2	No evidence of corrosion or spalling at post-tensioning or end fittings.	
____	8.1.5.3	No substantial leakage damage to roof deck.	
____	8.1.5.4	No damage to masonry and/or concrete elements due to freeze/thaw action.	
____	8.1.5.5	No damage to concrete surfaces due to chloride-laden concrete.	
	STRUCTURAL ELEMENTS		
____	8.1.5.6	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
____	8.1.5.7	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
____	8.1.5.8	No vertical strength discontinuities.	
____	8.1.5.9	The exterior concrete or masonry walls are anchored to each of the diaphragm levels.	
____	8.1.5.10	No cross grain bending or tension in wood ledgers.	
____	8.1.5.11	Wall reinforcing greater than 0.0025 Ag each way with a maximum spacing of 18 inches.	
____	8.1.5.12	No torsion.	
____	8.1.5.13	Metal deck has topping slab with a minimum thickness of 3 inches.	

True/
False | Comments

____ 8.1.5.14 Precast concrete diaphragms have a topping slab with a minimum thickness of 3 inches that is doweled into the walls.

____ 8.1.5.15 Chords around diaphragm openings greater than 50 percent of the width.

____ 8.1.5.16 Large tensile capacity at re-entrant corners or other plan irregularities.

FOUNDATIONS

____ 8.1.5.17 Wall panels have dowels into ground floor slab or foundation equal to vertical wall reinforcing.

____ 8.1.5.18 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

____ 8.1.5.19 The foundation is not composed of unreinforced masonry or stone rubble.

____ 8.1.5.20 There is no foundation or superstructure damage due to heaving soil.

____ 8.1.5.21 Building is not founded on a soil which is subject to liquefaction.

NON-STRUCTURAL ELEMENTS

____ 8.1.5.22 Cornices, parapets, and other appendages are reinforced and anchored to the structure.

____ 8.1.5.23 Exterior cladding and veneer are well anchored.

*See Section 6.10 (ATC-14 - Chapter 8, Section 8.1.5), for detailed discussion on each of these checklist issues.

Checklist 11. Precast Concrete Frame and Concrete Shear Wall Buildings*
(Low Seismicity Regions)

True/
False — Comments

MATERIALS

____ 8.2.5.1 No signs of significant deterioration in vertical or lateral force resisting system.

____ 8.2.5.2 No evidence of corrosion or spalling at post-tensioning or end fittings.

____ 8.2.5.3 No damage to masonry and/or concrete elements due to freeze/thaw action.

____ 8.2.5.4 No damage to concrete surfaces due to chloride-laden concrete.

STRUCTURAL ELEMENTS

____ 8.2.5.5 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 8.2.5.6 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 8.2.5.7 No vertical strength discontinuities.

____ 8.2.5.8 The exterior concrete or masonry walls are anchored to each of the diaphragm levels.

____ 8.2.5.9 Wall reinforcing greater than 0.0025 Ag each way with a maximum spacing of 18 inches.

____ 8.2.5.10 No torsion.

____ 8.2.5.11 No vertical mass or geometric irregularities.

____ 8.2.5.12 Walls are continuous to the foundation.

____ 8.2.5.13 Metal deck has topping slab with a minimum thickness of 3 inches.

True/ False			Comments
____	8.2.5.14	Precast concrete diaphragms have a topping slab with a minimum thickness of 3 inches that is doweled into the walls.	
____	8.2.5.15	If frame girders bear on corbels, length of bearing is greater than 3 inches.	
____	8.2.5.16	Chords around diaphragm openings greater than 50 percent of the width.	
____	8.2.5.17	Large tensile capacity at re-entrant corners or other plan irregularities.	
____	8.2.5.18	Diaphragm openings at walls are less than 25 percent of the length.	
____	8.2.5.19	Special reinforcement around all wall openings.	
	FOUNDATIONS		
____	8.2.5.20	Vertical wall reinforcing is doweled into the foundation.	
____	8.2.5.21	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.	
____	8.2.5.22	The foundation is not composed of unreinforced masonry or stone rubble.	
____	8.2.5.23	There is no foundation or superstructure damage due to heaving soil.	
____	8.2.5.24	Building is not founded on a soil which is subject to liquefaction.	
	NON-STRUCTURAL ELEMENTS		
____	8.2.5.25	Cornices, parapets, and other appendages are reinforced and anchored to the structure.	
____	8.2.5.26	Exterior cladding and veneer are well anchored.	

*See Section 6.11 (ATC-14 - Chapter 8, Section 8.2.5), for detailed discussion on each of these checklist issues.

Checklist 12. Reinforced Masonry Wall Buildings With Wood or Metal Deck Diaphragms*
(Low Seismicity Regions)

True/ False			Comments
		MATERIALS	
____	9.1.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
____	9.1.5.2	No evidence of corrosion or spalling at post-tensioning or end fittings.	
____	9.1.5.3	No substantial leakage damage to roof deck.	
____	9.1.5.4	No damage to masonry and/or concrete elements due to freeze/thaw action.	
____	9.1.5.5	No damage to concrete surfaces due to chloride-laden concrete.	
		STRUCTURAL ELEMENTS	
____	9.1.5.6	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
____	9.1.5.7	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
____	9.1.5.8	No vertical strength discontinuities.	
____	9.1.5.9	The exterior concrete or masonry walls are anchored to each of the diaphragm levels.	
____	9.1.5.10	No cross-grain bending or tension in wood ledgers.	
____	9.1.5.11	Total vertical and horizontal wall reinforcing greater than 0.0025 Ag with 0.0007 Ag minimum in either direction. Maximum spacing 48 inches. All vertical bars extend to the top of the wall.	
	9.1.5.12	No torsion.	
____	9.1.5.13	No vertical mass or geometric irregularities.	

True/
False

Comments

____ 9.1.5.14 Wall anchors spaced at 4 feet or less.

____ 9.1.5.15 Diaphragm openings at walls are less than 25 percent of the length.

____ 9.1.5.16 All wall openings have trim reinforcing on all sides.

FOUNDATIONS

____ 9.1.5.17 Vertical wall reinforcing is doweled into the foundation.

____ 9.1.5.18 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

____ 9.1.5.19 The foundation is not composed of unreinforced masonry or stone rubble.

____ 9.1.5.20 There is no foundation or superstructure damage due to heaving soil.

____ 9.1.5.21 Buildings taller than six stories are not founded on a soil which is subject to liquefaction.

NON-STRUCTURAL ELEMENTS

____ 9.1.5.22 Cornices, parapets, and other appendages are reinforced and anchored to the structure.

____ 9.1.5.23 Exterior cladding and veneer are well anchored.

*See Section 6.12 (ATC-14 - Chapter 9, Section 9.1.5), for detailed discussion on each of these checklist issues.

Checklist 13. <u>Reinforced Masonry Bearing Wall Precast Concrete Diaphragm Buildings</u>*
(Low Seismicity Regions)

True/
False | Comments

MATERIALS

____ 9.2.5.1 No signs of significant deterioration in vertical or lateral force resisting system.

____ 9.2.5.2 No evidence of corrosion or spalling at post-tensioning or end fittings.

____ 9.2.5.3 No damage to masonry and/or concrete elements due to freeze/thaw action.

____ 9.2.5.4 No damage to concrete surfaces due to chloride-laden concrete.

STRUCTURAL ELEMENTS

____ 9.2.5.5 Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.

____ 9.2.5.6 Building has a redundant lateral system insuring structural stability in the event of a single component failure.

____ 9.2.5.7 No vertical strength discontinuities.

____ 9.2.5.8 The exterior concrete or masonry walls are anchored to each of the diaphragm levels.

____ 9.2.5.9 Total vertical and horizontal wall reinforcing greater than 0.0025 Ag with 0.0007 Ag minimum in either direction. Maximum spacing 48 inches. All vertical bars extend to top of the wall.

____ 9.2.5.10 No torsion.

____ 9.2.5.11 No vertical mass or geometric irregularities.

True/
False | | Comments

True/False		
____	9.2.5.12	Topping slabs with a minimum thickness of 3 inches are continuous through interior walls and have dowels into exterior walls to match the slab steel.
____	9.2.5.13	Wall anchors spaced at 4 feet or less.
____	9.2.5.14	Diaphragm openings at walls are less than 25 percent of the length.
____	9.2.5.15	Large tensile capacity at re-entrant corners or other plan irregularities.

FOUNDATIONS

True/False		
____	9.2.5.16	Vertical wall reinforcing is doweled into the foundation.
____	9.2.5.17	In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.
____	9.2.5.18	The foundation is not composed of unreinforced masonry or stone rubble.
____	9.2.5.19	There is no foundation or superstructure damage due to heaving soil.
____	9.2.5.20	Building is not founded on a soil which is subject to liquefaction.

NON-STRUCTURAL ELEMENTS

True/False		
____	9.2.5.21	Cornices, parapets, and other appendages are reinforced and anchored to the structure.
____	9.2.5.22	Exterior cladding and veneer are well anchored.

*See Section 6.13 (ATC-14 - Chapter 9, Section 9.2.5), for detailed discussion on each of these checklist issues.

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NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

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NCEER-87-0025 "Proceedings from the Symposium on Seismic Hazards, Ground Motions, Soil-Liquefaction and Engineering Practice in Eastern North America," October 20-22, 1987, edited by K.H. Jacob, 12/87, (PB88-188115/AS).

NCEER-87-0026 "Report on the Whittier-Narrows, California, Earthquake of October 1, 1987," by J. Pantelic and A. Reinhorn, 11/87, (PB88-187752/AS). This report is available only through NTIS (see address given above).

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NCEER-88-0001 "Workshop on Seismic Computer Analysis and Design of Buildings With Interactive Graphics," by W. McGuire, J.F. Abel and C.H. Conley, 1/18/88, (PB88-187760/AS).

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RECOMMENDED MODIFICATIONS TO ATC-14

by

C.D. Poland[1] and J.O. Malley[2]

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