NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

A Procedure for the Seismic Evaluation of Buildings in the Central and Eastern United States

by

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NOTICE

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This report is based upon the methodology developed in
ATC-14 - Evaluating the Seismic Resistance of Existing Buildings

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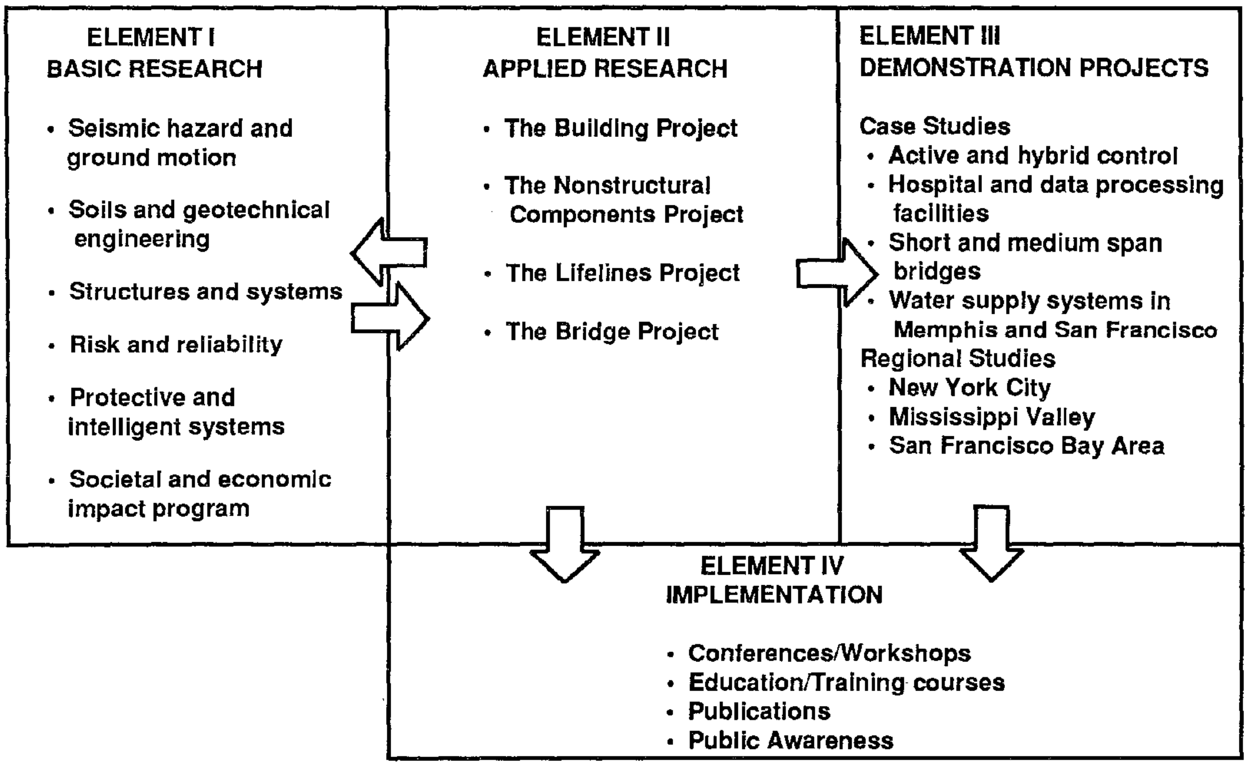
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

Two of the short-term products of the **Building Project** will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The **structures and systems program** constitutes one of the important areas of research in the **Building Project**. Current tasks include the following:

1. Continued testing of lightly reinforced concrete external joints.
2. Continued development of analytical tools, such as system identification, idealization, and computer programs.
3. Perform parametric studies of building response.
4. Retrofit of lightly reinforced concrete frames, flat plates and unreinforced masonry.
5. Enhancement of the IDARC (inelastic damage analysis of reinforced concrete) computer program.
6. Research infilled frames, including the development of an experimental program, development of analytical models and response simulation.
7. Investigate the torsional response of symmetrical buildings.

The evaluation of the performance of existing buildings ranges from relatively rapid assessment to detailed analysis. In the former category, the present report provides an important tool for engineers to evaluate buildings typical in the east and midwest United States. It relies on ***ATC-14 - Evaluating the Seismic Resistance of Existing Buildings*** *and is intended for use in regions of moderate seismicity. Some of the material builds on research that has been underway at NCEER, which has been completed prior to the publication of this report. The results of these projects are summarized in other NCEER reports.*

ABSTRACT

In January 1983, the National Science Foundation awarded the Applied Technology Council a 3-year grant to develop methods for evaluating the seismic strength of existing buildings. The objective of the project was to develop a comprehensive practical methodology that could guide engineers throughout the United States in evaluating existing buildings to determine potential earthquake hazards and identify buildings or building components that present unacceptable risk to human lives.

The Methodology developed for this project, titled "ATC-14 - Evaluating the Seismic Resistance of Existing Buildings", was developed for the Applied Technology Council by H.J. Degenkolb Associates, Engineers. A Project Engineering Panel of eight prominent structural engineers participated throughout the project to ensure that a consensus based methodology was developed.

The ATC-14 procedure for the seismic evaluation of existing buildings has begun to gain wide acceptance since its publication in 1987. In 1988, the National Center for Earthquake Engineering Research (NCEER) funded a project to critically assess the applicability of ATC-14 to buildings in the Eastern and Central United States. This NCEER project developed a large volume of recommended modifications to ATC-14 procedure's recommendations for the seismic evaluation of buildings in regions of low seismicity (NCEER Report No. 89012). This NCEER report is for a second project to produce a separate document for the seismic evaluation of existing buildings which specifically focuses on structures in these areas of the country. This report should prove to be a valuable tool for practicing engineer performing seismic evaluations on buildings in the Eastern and Central United States.

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SYMBOLS AND DEFINITIONS

The following symbols and definitions apply to the methodology provided in this report:

a = Ground acceleration

A = Maximum pseudo acceleration of the mass of the oscillator

A_a = Effective peak acceleration (also defined as EPA)

A_{br} = 1.5 times the average area of a diagonal brace

A_c = Summation of the cross sectional area of all columns in the story under consideration

A_v = Effective peak velocity-related acceleration

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

C = Numerical coefficient determined from Equation 4.4

C_n = Overburden Pressure Correction Factor

C_p = Horizontal force factor given in Table 4.8

C_t = Numerical coefficient as defined in Section 4.4.3.B.1

C/D = Capacity/Demand Ratio

CSR = Cyclic stress ratio

CCSR = Critical cyclic stress ratio

d = Ground displacement

d_b = Bar diameter

D = Maximum relative displacement between the mass of the oscillator and its base

D_d = Diaphragm depth (depth of the building parallel to the direction along which analysis forces are applied)

D_s = Length, in feet, of shear wall or braced frame segments in the direction parallel to the applied forces

Dsmax = Length, in feet, of the longest shear wall or braced frame segment

DL = Dead load

E = Seismic load

E_r = Energy ratio corrected value

f_i = Distributed portion of a total lateral force at level i

F = Multiplication factor of map acceleration values for different return periods

F_i, F_x = Distributed force applied to level i or x

F_p = Lateral force on an element or component of the structure

F_{px} = Lateral force on floor or roof diaphragm

F_t = Concentrated force at the top of the structure

F_{va} = Scaling factor to modify velocity-to-acceleration ratios for differing recurrence intervals, determined from Equation 3.2

g = Acceleration due to gravity

h = Story height, in inches

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

H/D = Height-to-depth ratio of wall under consideration

I = Importance factor given in Table 4.7

k_b = Moment of inertia of beam, in inches^4, divided by center to center length between columns, in inches

k_c = Moment of inertia of column, in inches^4, divided by center to center length between beams, in inches

j = Story level under consideration

KL/r = Effective slenderness ratio of element under consideration

l_w = Total length of shear walls under consideration

L = Diaphragm span length, in feet

L_1 = Equivalent diaphragm span length, in feet, determined from Equation 4.14

L_{br} = Average length of diagonal braces

LL = Live load

M_s = Surface wave magnitude of earthquake

n = Total number of stories above ground level

n_c = Total number of columns at story level

n_f = Total number of frames in the direction of loading

N_{br} = Number of braces in tension in the direction of loading

N = Numerical coefficient based on SPT

N_cz = Corrected N value

R_w = Structural response modification factor given in Table 4.4

r_d = Soil flexibility reduction factor

s = Average span length of braced spans

S = Site coefficient for soil characteristics given in Table 4.5

S_a = Response spectral acceleration

S_d = Response spectral displacement

S_v = Response spectral velocity

SPT = Standard Penetration Test

T = Undamped natural period of vibration, in seconds

T_l = Return period, in years

t = Wall thickness

U = Required strength to resist factored loads

v = Ground velocity

v_u = Yield capacity of diaphragm as defined in Section 4.4.6

V = Base shear

V = Maximum pseudo relative velocity

V_{AVG} = Average wall shears

V_c = Average shear in each column

V_{cw} = Total yield capacity of crosswalls (v_u times the length of the crosswalls)

Vj = Maximum story shear at story level j

$w_i w_x$ = Portion of W which is located at or assigned to level i or x

w_{px} = Weight of floor or roof diaphragm

W = Total seismic dead load

W_d = Total dead weight tributary to diaphragm, including walls perpendicular to the direction of motion

W_j = Total seismic dead load of all stories above level j

W_p = Weight of a portion of a structure or nonstructural component

W_w = Wall weight at open front of building

β = Percent critical damping

δ_i = Elastic deflection at level i relative to the base

Δ = Story drift at any level

τ = Cyclic shear stress applied to ground

σ_o = Total overburden stress at depth of concern

SECTION 1

INTRODUCTION

1.1 Background

Earthquake engineering has developed during the last five decades at a steadily increasing rate. Research results from many universities and research centers, lessons learned from studies of ground motion records and earthquake effects on structures, and other technological developments have been continuously adopted into practice and incorporated into new codes and standards. Nevertheless, many existing buildings in seismically active regions of the United States were designed and constructed in accordance with past codes, standards, and practices that are technically obsolete today and are considered inadequate. Although many of these buildings may perform well in future earthquakes owing to the skill and farsightedness of their designers, a great many others are likely to fail dangerously, causing many deaths and injuries.

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 the 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 these areas of 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.

Although recent improvements in seismic design codes and standards have resulted in increased earthquake resistance for newer properly designed buildings, they have created a dilemma for the older existing ones. Because they were constructed according to "lesser" codes and standards (by today's standards) does that mean they are all dangerous? Can we identify those buildings that are likely to be damaged to an extent that would create unacceptable life-safety hazards for their occupants? Similarly, can we identify those buildings that, although they do not satisfy current design codes and standards, would be expected to perform satisfactorily in the next earthquake?

These questions are faced daily by building owners, occupants, and governmental officials. Consideration of the financial issues, such as the potential economic losses due to earthquake damage and the cost of structural upgrading, complicates the problems further and makes decision making extremely difficult. Clearly, procedures and guidelines are needed to evaluate existing buildings to identify those that are seismically hazardous, that is, those that could endanger human lives in an earthquake that can reasonably be expected to occur during the useful life of the building.

1.2 Objectives, Concepts, and Limitations

The overall objective of the ATC-14 Project was to develop a comprehensive but practical methodology that could guide engineers in all seismic zones of the United States in evaluating existing buildings to determine potential earthquake hazards and identify buildings or building components that present unacceptable risk to human lives. The methodology presented herein fully addresses this objective.

The methodology assumes that a "hazardous building" endangers human lives in an earthquake if one or more of the following events occurs:

- The entire building collapses
- Portions of the building collapse
- Components of the building fail and fall
- Exit and entry routes are blocked, preventing the evacuation and rescue of the occupants

Therefore, the identification of life-safety hazards in an existing building consists of determining whether any of these events could potentially happen for that building during an earthquake that could reasonably be expected to occur during its lifetime. Hence, a major portion of the methodology is dedicated to directing the evaluating engineer on how to determine if there are any weak links in the structure that could precipitate structure or component failure. Potential weak links have been identified from detailed reviews of building performance data from past earthquakes. On the basis of this information, procedures have been developed to first determine if any of the identified potential weak links exist in the building under consideration, and then to evaluate whether any of the potential weak links found to exist could indeed present a life-safety hazard.

Each of the possible weak links or problems that have been identified therefore becomes part of a list of items to be considered in the seismic evaluation. This list of items should result in more consistency in the seismic evaluation process because every engineer will perform the evaluation with the same set of "ground rules". But, the manual is not intended to be used as a rigid document with no room for interpretation by the engineer. Rather, it is intended to alert the engineer to the building features that could present life-safety hazards for that seismic risk zone. For each building being studies, it will always be the engineer's responsibility to determine the applicability and to perform an appropriate evaluation of each item of concern.

The ground motion criteria specified in this methodology utilize the effective peak acceleration and effective peak velocity maps developed in the ATC-3 project (ATC, 1978). The local ground motion (ground motion for a specified site), however, is defined by a set of response spectra corresponding to the mean-value spectrum shapes as opposed to the mean-plus-one-sigma shapes used in the ATC-03 project. The extra conservatism that can be justified for new buildings was not believed necessary for existing buildings. Rather, the aim of this project was to develop a methodology to evaluate the effect of the most probable large earthquake event to which a structure would be subjected within a specified lifetime. Similarly, special effort was made in prescribing procedures for estimating building capacities to be as realistic as possible with regard to the material properties and analytical methods.

The methodology is intended to be used as a guide by structural engineers experienced in seismic design and analysis of buildings. The methodology is general in nature and will enable the evaluating engineer to identify weak links known to represent life-safety hazards. In rare cases, however, there may be peculiar characteristics of specific buildings that may ultimately cause their failures in earthquakes, but that may not have been observed in past earthquakes or as a result of research investigations. In these cases, the weak link(s) may not be revealed by application of this methodology, and the evaluating engineer will have to carefully scrutinize the structure to identify such problems and give careful consideration to their resolution.

The ATC-14 methodology reflects both the state of the art and the state of the practice. It is based on an extensive research effort and numerous discussions and deliberations between the project subcontractor and the advisory Project Engineering Panel (Appendix A). Many parts of the methodology, including the factors that cause buildings to be seismically hazardous, reflect the consensus of this group.

1.3 Organization and Main Components of the Methodology

The methodology consists of background material (Section 2), seismic loading criteria (Section 3), general evaluation methodology (Section 4), evaluation methodology for specific types of buildings (Sections 5 through 10), and evaluation methodology for nonstructural elements (Section 11). Also included as Section 12 are several case studies illustrating application of the methodology.

Prior to using the structural evaluation methodologies for specific building types (Sections 5 through 10), which are at the heart of the methodology, it is recommended that the user familiarize himself/herself with Sections 2 through 4. Section 2 describes the results of a state-of-the-practice review performed by the Subcontractor and includes a summary of a literature search, review of the ATC-3-06 document (ATC,1978) trial design program, an analysis of the Phase 2 reports done in the U.S. Navy's seismic evaluation program, and the results of an intensive interview effort in which consultants nationwide, experienced in the seismic evaluation of buildings, were queried to determine both the basic evaluation approaches followed in today's practice and any special techniques that may be used by the different structural offices. In addition, the NCEER Review developed a good deal of background information, including an identification of NCEER projects relevant to ATC-14, a discussion of regional differences between the western United States and the rest of the country, and a listing of the status of building code adoption in each state. Section 2, in general, provides insight into the reasons for adopting the basic approach used in the ATC-14 building seismic evaluation methodology.

In Section 3, developed by the Seismic Loading Consultant, procedures for determining the appropriate loading criteria for the site being evaluated are presented. Included are a review and evaluation of existing zonation maps, procedures for developing recommended site-specific response spectra for existing buildings (damping assumed to be 5% of critical; probability of nonexceedance specified as approximately 90% in 50 years), procedures for developing response spectra for different damping levels and different probability levels, and sample computations. As previously indicated, the seismic loading criteria are based on the loading criteria developed for the ATC-3 project (ATC, 1978). The NCEER project developed a discussion of the seismicity issues for the Eastern United States, and the potential for damaged caused by secondary seismic hazards.

Section 4, developed by the project Subcontractor, contains a detailed description of the general methodology recommended for the seismic evaluation of existing buildings. This Section includes a step-by-step description of the procedure to be followed when evaluating a specific building:

- Data collection procedures, including a review of existing drawings and reports, field investigation of existing conditions, and testing methods

- Descriptions of the model building types adopted in the methodology and procedures for determining which of these model building type(s) are most appropriate for the building being evaluated

- Detailed analysis procedures for various building types and components

Included in the section on analysis procedures are methods for calculating member capacities, rapid analysis procedures to determine overall building strength and interstory drift, an equivalent lateral force procedure, a procedure for using dynamic analysis, a methodology for determining lateral forces on elements of structures and nonstructural components, and a special analysis procedure for buildings with wood diaphragms. Included also is a flow chart that outlines the steps to be followed in evaluating an existing building.

Sections 5 through 10 contain procedures specifically applicable to the 15 model building types identified for this project. These model building types, which include all major types of construction materials, were developed to incorporate as many of the structural and performance characteristics of the total nationwide building inventory as possible. Following are the construction materials considered: Wood (Section 5), Steel (Section 6), Cast-in-Place Concrete (Section 7), Precast Concrete (Section 8), Reinforced Masonry (Section 9), and Unreinforced Masonry (Section 10). Each of these Sections contains:

- A description of the model building type

- Performance characteristics

- Examples of the performance of like buildings in past earthquakes

- Description of expected loads and load paths

- Procedures for evaluating buildings in regions of low seismicity (regions zoned for an expected effective peak acceleration (EPA) $\leq$ .10 g)

The procedures for evaluating buildings consist of a collection of statements with a related concern and suggested specific analysis technique if further study should be necessary. Each statement relates to a vulnerable area in the structural system that requires specific consideration.

The evaluation statements are written such that a positive or "true" response to a statement implies that the building is adequate in that area. If a building then passes all the related statements with true responses, it can be passed without further evaluation. It must once again be stressed that these evaluation statements are intended to flag areas of concern for the evaluating engineer. The final decision regarding adequacy, need for further study, or need for strengthening still rests with the engineer, regardless of the statements. For these reasons, the evaluation procedures, even the initial screening procedures, must be applied by a knowledgeable structural engineer or at least under his or her supervision.

Each statement carries with it a concern that explains in commentary style why the statement was written. These concerns are intended to further assist the evaluating engineer in dealing with the issue stated. Addressing the concern should therefore take precedence over the specific statement.

For statements that are "false", additional evaluation is required. This does not necessarily imply that a complete structural evaluation is necessary, or that the building is automatically deficient. In fact, the suggested procedure limits the evaluation to only the area of concern. It is offered as a suggested procedure because the responsibility for the evaluation rests with the structural engineer, who may elect to perform an alternate evaluation procedure. This is permissible as long as it addresses and leads to an opinion regarding the issue raised in the statement. Deficiencies are identified only after an appropriate detailed evaluation has been made.

The evaluation procedures suggested are based on the ATC-3 and ATC 6-2 type approaches to considering the capacity of the element under review and the demand placed on that element. A recommended Capacity/Demand (C/D) ratio is listed for each statement, based on the anticipated excess capacity available in the element and the level of overall system ductility assumed in the demand criteria. The recommended C/D ratio can be compared to that calculated for the building being evaluated. In this way not only can the weak links in the structural system be identified, but also their ranking and expected severity can be estimated.

Sections 5 through 10 have been designed to be utilized in conjunction with Sections 3 and 4, but otherwise to stand alone. Hence, in those instances where specific types of life-safety hazards and corresponding weak links are similar for more than one model building type, the issues and arguments are repeated for each applicable building type. This approach, although repetitive, was adopted to facilitate use of the document (i.e., to enable the evaluating engineer to focus on one Section corresponding to the applicable building type, rather than having to constantly refer to numerous generalized sections of the report).

Section 11, which is devoted to the evaluation of nonstructural elements, is structured similarly to Sections 5 through 10. This Section received a major expansion as a result of the NCEER review. Following Section 12, which contains illustrative examples pertaining to buildings in regions of high seismicity, are several appendices providing supplementary information. Appendix A contains a list of project participants; Appendix B, results of the interview/questionnaire process involving consultants nationwide; Appendix C, abbreviated building evaluation checklists for each of the 15 model building types; Appendix D, a preliminary procedure for the evaluation of liquefaction potential; and Appendix E, mailing addresses for reference standards listed in Section 4.4.1.

SECTION 2

STATE OF PRACTICE REVIEW AND OTHER BACKGROUND INFORMATION

One major task of the original ATC-14 project was the state-of-the-art/practice review, which was felt to be necessary for two reasons. First, in recent years, there has been a major effort to advance the state of the art in seismic evaluation procedures for existing buildings, and this report should consider and incorporate any appropriate information that can be gained from these recent studies. Second, an effort to determine the present state of the practice of engineers who perform seismic evaluations was felt to be critical to the development of this methodology. Without an accurate impression of the state of the practice, it would be difficult to develop evaluation procedures that will be useful to the design profession.

During the development of the NCEER project, which reviewed the ATC-14 document, a number of other areas of background information which could be useful to the evaluating engineer were identified. This information was developed into sections or Sections of the original NCEER report. These sections have been modified for inclusion into the evaluation document to provide the evaluating engineer with supplemental information which could be of assistance in the seismic evaluation.

The state-of-the-art/practice review involved three major efforts: (1) a literature survey, (2) a review of ATC-3-06 (ATC, 1984) trial designs and U.S. Navy evaluation reports, and (3) consultant interviews. The following three topics of background information were developed: 1) identification of NCEER projects relevant to ATC-14, 2) regional differences between the Western United States and the rest of the country, and 3) the status of building code adoption by each state. The following sections present a summary of the basic findings of the state-of-the-art/practice review, and the background information developed in the NCEER review.

2.1 Literature Survey

A major objective of the literature survey was to identify information that could be useful in developing the building seismic evaluation methodology. Consequently, the literature survey focused on a review of (1) earthquake damage reports; (2) existing and proposed code provisions; (3) previously developed seismic evaluation methodologies; (4) reports on analytical and experimental research, including a special focus on the various materials utilized in building structural systems; and (5) testing methods.

Earthquake damage reports considered include those for California earthquakes such as San Francisco in 1906 (ASCE, 1907; Committee on Fire and Earthquake Damage to Buildings, 1907; State Earthquake Investigation Committee, 1969; USGS, 1907), Kern County in 1952 (CDMG, 1955; Degenkolb, 1955; SEAONC, 1952; SEAONC Lateral Force Committee, 1953; Steinbrugge and Moran, 1954), and San Fernando in 1971 (Lew et al., 1971; Mahin et al., 1976; Murphy, 1973; Steinbrugge et al., 1971); Alaska in 1964(C&GS, 1967; NRC, 1973); and Managua, Nicaragua in 1972 (EERI, 1973a,b; Wyllie et al., 1974). These reports proved to be an invaluable source for identifying and cataloging the performance characteristics of the different building types. Through their identification of characteristics that constitute life-safety hazards, these damage reports formed the basis for the entire methodology developed under this project.

Existing and proposed code provisions reviewed include the provisions developed for government agencies such as the Veterans Administration (Veterans Administration, 1977), the State of California (Department of General Services, 1979), the Department of the Army (URS/John A. Blume and Associates, Engineers, 1984), and local municipalities, as well as foreign codes such as New Zealand's concrete requirements (Standards Association of New Zealand, 1982). The City of Los Angeles provisions covering existing buildings (City of Los Angeles, 1985), for example, were especially helpful in developing the evaluation procedures for unreinforced masonry structures. In general, the provisions reviewed provided information useful in developing both the preliminary and detailed evaluation procedures.

Previously developed seismic evaluation methodologies reviewed include those developed by federal agencies such as the National Bureau of Standards (Culver et al., 1975), the U.S. Navy (Freeman, 1982), and the General Services Administration (GSA, 1976), as well as methodologies developed in other countries, such as the Japanese rapid evaluation procedures for existing concrete buildings (Aoyama, 1981; Hirosawa, 1981). Most of these and other previously developed procedures call for combining many different building characteristics into a single, often numerical, rating for the building. By determining a single rating, however, these methodologies may mask individual life safety hazards. Although certain of these methodologies could be useful in ranking the seismic hazard of a large inventory of buildings, they did not seem appropriate for the evaluation of individual buildings. The items considered in the different methodologies, however, did provide useful information concerning the major topics that should be considered in seismic evaluations of individual buildings.

The review of papers and reports dealing with general analysis techniques and strengthening/retrofit procedures provided information on detailed analytical modeling and element behavior. Papers/reports reviewed include the ATC-10 report (ATC, 1982) and seminar papers describing strengthening procedures typically used in the United States and Japan (Hanson, 1980, 1981, 1982). This information was useful in the establishment of modeling techniques and member-capacity determination procedures developed for this project.

Reports on the response of structural systems composed of various materials (wood, steel, concrete, prestressed concrete, masonry, and unreinforced masonry) provided extremely useful information for the development of the evaluation procedures. Reports reviewed include those on (1) the response of wood walls, diaphragms, and connections; (2) analytical and experimental research results on steel bracing elements and connection details conducted at the University of California at Berkeley, the University of Michigan, and Lehigh University; (3) analytical methods for the nonlinear response of steel-framed buildings; (4) demand-to-capacity ratios, such as those proposed in New Zealand and those provided in the ATC-6-2 Report, Seismic Retrofitting Guidelines for Highway Bridges (ATC, 1983); (5) preliminary analyses of tilt-up wall structures, as drawn from information presented by the Structural Engineers Association of Southern California and Agbabian Associates; (6) the New Zealand provisions for the design of prestressed concrete frames; (7) detailed analyses of infilled walls conducted by researchers from the University of California at Berkeley and the University of Michigan; and (8) the ABK joint venture research on unreinforced masonry bearing wall buildings (ABK, 1984) that provide the basis for both the preliminary and detailed evaluation procedures for this class of buildings.

Testing methods reviewed were restricted primarily to those appropriate for the data collection phase of the seismic evaluation procedure. Most of this work concerns the nondestructive testing of concrete and masonry elements.

2.2 Review of Trial Designs and Evaluation Reports

This phase of the state-of-the-practice review consisted of two major efforts: (1) a review of the trial design program conducted by the Building Seismic Safety Council (BSSC) to test the efficacy of the Amended ATC-3-06 document, Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC, 1984), and the cost of its application; and (2) an analysis of the Phase 2 reports done in the U.S. Navy's seismic evaluation program.

2.2.1 BSSC Trial Design Review

In 1982 the Building Seismic Safety Council, under the sponsorship of the Federal Emergency Management Agency, initiated a trial design program to test the amended ATC-3-06 Tentative Provisions. The program was intended to determine the impact of the provisions on construction costs throughout the country and to evaluate the clarity, usability, and completeness of the document. Nine trial design locations were chosen to cover the range of seismicities encountered across the nation. A total of 17 subcontractors were selected to perform trial designs on 46 buildings selected to encompass a wide range of typical modern construction types. At the conclusion of the trial design program, BSSC revised and updated the ATC-3-06 document and published it as the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings (BSSC, 1985).

The reports and calculations submitted by the BSSC subcontractors were reviewed under this project to gain insight into the national state of practice for seismic design and to obtain information that would be useful in developing the methodology for evaluating the seismic strength of existing buildings. The first conclusion that can be drawn from reviewing the BSSC trial design reports is that the nationwide group of engineers involved in the trial design program were receptive to the approach presented in ATC-3-06 (ATC, 1984) and could apply the provisions. It did appear, though that the engineers from areas of low seismicity took more effort to properly interpret the provisions than did the subcontractors who routinely consider seismic forces. The subcontractors from areas of highest seismicity typically gave more suggestions and recommendations for changes to the provisions than did engineers from areas of low seismicity. It is also interesting to note that most of the subcontractors from areas of low seismicity believed that the ATC 3-06 provisions should become code regulations. Engineers from highly seismic regions, on the other hand, typically believed that the document should become recommended provisions, but not code regulations.

Examination of the calculations prepared by the different BSSC subcontractors showed that a similar basic thought process is followed in the preliminary stages of seismic design. Because the trial design buildings were regular, no special analysis techniques (e.g., dynamic analysis) were used to calculate the preliminary sizes. The designs did not require the preparation of working drawings; therefore, the calculations done for connection details were limited. As a result, it was difficult to determine if the designs contained details that could present life-safety hazards. Some of the subcontractors did show sketches of typical connection details. The details shown for steel and cast-in-place concrete structures appeared to be capable of providing some ductility. Some of the details for the connection of precast elements provided for structures in regions of low seismicity, however, may not provide ductility, even though they may be adequate for the code force levels.

The review of the BSSC trial designs of the amended ATC-3-06 Tentative Provisions provided an indication of the large national variation in the state of practice for seismic design. The review also provided insight into the typical calculation procedures followed by different subcontractors from various regions of the country as well as the amount of effort required to properly implement the provisions. The information provided by this review was useful in directing the focus and content of the methodology prepared under this project.

2.2.2 Navy Phase II Evaluation Review

During the past few years, the Department of the Navy has conducted seismic evaluations of Naval Facilities on the West Coast and in the Pacific region. The first phase of these evaluations consisted of preliminary screening and use of the Navy's Rapid Seismic Analysis Procedure (Freeman, 1982) to estimate the expected damage level. The results of the Phase I studies led to detailed structural evaluations, known as Phase II evaluations.

As part of the state-of-the-practice survey, detailed Phase II structural evaluations performed by four California-based consultants on 75 structures at 13 different locations were reviewed. The purposes of this review of Navy Phase II evaluations were: (1) to test the building classification and identification system developed for this project; (2) to compare the loading criteria, analysis techniques, and capacity determination methods of the different consultants; and (3) to study the problems and concerns identified by each consultant as a result of each evaluation. The results of this review were useful in refining the proposed building classification system and in determining the proper detailed seismic evaluation procedures.

Building Classification and Identification. The four consultants involved in this review considered various types of building material. In most cases, the 75 structures evaluated could be categorized under one of the 15 building types proposed for this project/methodology. This review also showed that (1) in some cases, judgment is required to determine if a building containing shear walls with large openings will act primarily as a shear-wall building or a frame structure; and (2) it is necessary to give special consideration to building additions that rely on the lateral resistance of the original structure. The latter finding suggests that some structures may have dual lateral force resisting systems, neither of which have sufficient capacity to be considered the primary system. This finding was considered in the building identification system developed for this project.

Loading Criteria. For each base, the Navy provided site specific response spectra (Navy spectra) with a 20% chance of exceedance in fifty years. One consultant (Consultant 1) modified the spectra provided so that the evaluation could utilize the provisions of ATC-3-06 (ATC, 1984). Two of the consultants (Consultants 2 and 3) used two levels for their seismic evaluations: a design level earthquake specified by the Navy spectra, and a working stress level earthquake that was one-half of the Navy's design forces.

The damping values used by each consultant also varied. The consultants who evaluated the structures using two earthquake levels (Consultants 2 and 3) used a 5% damped spectrum for the "working stress" earthquake and 10% for the design level earthquake. The consultant who modified the Navy spectra (Consultant 1) to be compatible with the ATC-3-06 provisions used a value of 5% damping. Consultant 4 used a 5% damping ratio for structures that remained below the working stress level and 10% damping for buildings where a majority of the elements were stressed above working stress levels. This consultant doubled these values for wood-frame buildings.

Analysis Techniques. The analysis techniques employed by the consultants demonstrated a wide range of complexity. Consultant 1 used equivalent lateral force procedures for most of the buildings studied. This consultant used modified reduction factors to convert ATC 3-06 R factors to working stress levels. The base shear was distributed over the height of the structure according to ATC-3-06 provisions except for irregularly shaped buildings. In such structures, building modal analysis was used and proportioned to match the base shear from an equivalent lateral force procedure. The story forces were distributed to the frame or wall elements according to their elastic rigidity or on a tributary area basis, depending on the diaphragm stiffness.

Consultant 2 employed more complicated analysis procedures than Consultant 1. For wood structures, equivalent lateral force procedures were employed. For regular structures, the fundamental period was calculated using a Rayleigh procedure and the loads were distributed to the elements according to their elastic rigidities, assuming a rigid diaphragm, or by tributary area. Three-dimensional lumped mass modal analyses were performed for irregular structures. Modal combinations were typically done by the square-root-of-sum-of-squares (SRSS) method, although the complete-quadratic-combination (CQC) method was employed if the structure had closely spaced modes. Analysis of the structure for the design-level earthquake employed the Reserve Energy Technique first proposed by Blume (1960). In this technique, an approximate formula is used to reduce the stiffness of elements that are expected to yield. Successive iterations are required until the modified stiffnesses converge to the previously calculated value. This procedure results in local ductility demand estimates for the frame or wall elements.

The analysis procedures employed by Consultant 3 closely resembled those of Consultant 2. Modal analyses were performed for all structures except those of wood or unreinforced masonry construction. Lumped mass models were employed and diaphragm stiffness was modeled when necessary. SRSS modal combination was employed. Analysis of the design-level earthquake also employed the Reserve Energy Technique.

Consultant 4 used different levels of complexity in modeling techniques. For symmetric structures with rigid diaphragms, a single vertical element with lumped masses at the floors was analyzed. The stiffness of the wall and frame elements were combined into a single line element using static condensation procedures. For structures where diaphragm flexibility was important, but displacement perpendicular to the direction of motion were negligible, a more sophisticated model was constructed. The most general technique considered both the effects of diaphragm flexibility and torsional motion. For important or highly irregular structures, Consultant 4 used dynamic tests to determine the mode shapes and frequencies of the building. These results were used to construct the analysis model employed to check the member stresses.

Capacity Determination. The load factor equations used by the consultants were similar, though the coefficients varied somewhat. Consultant 1 employed the ATC-3-06 load combinations, with a special factor to be applied to brittle systems. Consultants 2 and 3 used almost identical combinations. Consultant 4 did not use a load combination to check column uplift. The earthquake forces used in the load factor equation analyzed by Consultant 4 were the result of the suggestion of Newmark and Hall (1982) to use an SRSS type combination of the vertical seismic force and horizontal loads in two orthogonal directions. The vertical seismic force was taken to be two-thirds of the horizontal loads. The allowable stress and member capacity values of all consultants were based on code formulas. Consultant 1 used working stress levels presented in the Uniform Building Code. Consultants 2 and 3 used working stress equations for the Level 1 earthquake, and code capacity values for the design earthquake (Level 2).

Consultant 4 used American Concrete Institute (ACI) code equations for all concrete elements except shear walls with good boundary elements and corner details. These elements were given shear capacities greater than $2\sqrt{f'_c}$.

The drift values checked by the consultants varied depending on the intensity of the earthquake and the importance of the building. The values ranged from .005 times the story height for "Mission Essential" buildings under the Level 1 earthquake to .015 times the story height for "Life-Safety" buildings under the Level 2 earthquake. Consultant 4 did not check story drifts, but did consider P-Δ effects.

Problems and Concerns. Comparisons by the four consultants of the demand and capacity for each of the elements in the structures evaluated resulted in the identification of areas of concern. The lack of strength and/or ductility of the connections between major elements, for example, was a major concern of the first three consultants. Diagonal bracing and diaphragm-to-exterior-wall connections were also often cited as areas of concern. All of the consultants encountered buildings with frame elements, shear walls, and diaphragms that did not meet the loading criteria. The lack of adequate ties in concrete columns that was an especially prevalent problem. The short column condition caused by partial-height shear walls was cited as a problem by three of the consultants. Another common problem was the lack of capacity of diagonal bracing elements. The problems resulting from torsional displacements and drift were mentioned by two of the consultants. The possibility of pounding between structures without adequate separation was mentioned for one structure. One consultant cited the omission of collector elements and trim reinforcement around wall and diaphragm openings.

In general, the review of the Navy Phase II evaluation reports provided critical insight into the practice currently followed for the seismic evaluation of existing buildings. The information taken from this review was useful in refining the building identification system, developing the detailed analysis procedures, and focusing the methodology on the appropriate areas of concern.

2.3 Consultant Interviews

The final phase of the state-of-the-practice review involved interviews with key consultants nationwide who have performed seismic evaluations of existing buildings. This process included personal interviews with 9 firms and the distribution of questionnaires to 15 others. The participants in the interview/questionnaire process included members of the following groups: (1) members of the ATC-14 Project Engineering Panel and other project participants, (2) BSSC Trial Design subcontractors, and (3) other consultants known to have performed seismic evaluations or who had expressed interest in the project.

2.3.1 Questionnaire/Interview Results

The questionnaire developed for the interviews was designed to determine the basic approach followed and any special techniques that may be used by the different structural offices. The major categories addressed by the questionnaire include the following: (1) general information, (2) information gathering, (3) preliminary analysis techniques, (4) detailed analytical techniques, and (5) other items. An effort was made to develop questions that could have simple answers and to encourage the consultants to provide a short explanation. In general, the questionnaire was designed to encourage the various consultants to discuss their own seismic evaluation practices, rather than to see if their methods were compatible with the proposed methodology.

General Information. In responding to this section of the questionnaire, typical consultants estimated that approximately 10-15 percent of their business deals with evaluation and/or retrofit of buildings for seismic hazards. These percentages varied from a low of 1 percent to a high of 75 percent. The three types of client seeking this information are: (1) government or public agencies, (2) developers or owners, and (3) architects. Only one of the consultants surveyed uses a written procedure to perform seismic evaluations. All the consultants treated buildings of different materials and/or type of construction separately. Most of the consultants were familiar with previously developed methodologies, but only use them if required to do so by the client. The results from foreign and U.S. research work are not typically used by most of the consultants. Many of the consultants mentioned that they have studied the ABK methodology (ABK, 1984) for unreinforced masonry buildings. The consultants agreed that more research information would be useful if it were synthesized into more useful forms.

Information Gathering. The questionnaire section on information gathering was designed to determine the types of information used to perform seismic evaluations. The consultants indicated that the first step in gathering information is a walk-through tour of the building to determine the general type and condition of the structural system. Tests of shear and compression strength of concrete and masonry elements are performed inmost cases,but only a small number of the consultants use more elaborate testing procedures. Verification of existing structural drawings is done by all the consultants, but the degree to which this is done varies depending on the completeness of the documents. If not structural drawings are available, some plans are prepared from field investigations, usually concentrating on the critical elements. About two-thirds of the consultants indicated they at least examine available original calculations; the rest preferred to prepare a complete set of their own calculations to be sure that the evaluation results would not be biased by the original design.

Preliminary Analysis Methods. The consultants indicated that a first level of analysis is performed to identify the structural system and any possible life-safety hazards. Although this stage of the evaluation concentrates on qualitative characteristics, most of the consultants perform some calculations to calibrate their judgment within an order-of-magnitude stress level. Decisions concerning adequacy and recommendations for further study are based on judgment and experience from the performance of similar buildings in past earthquakes. Many of the consultants stated that they would be reluctant to judge certain building types of the adequate without a more detailed review.

Detailed Analytical Techniques. Most of the consultants use code level forces (or some fraction thereof) to determine the seismic loads. To evaluate the capacity of the existing structure, approximately one-quarter of the consultants have factored up the working stress code level forces, often by a factor of about three. Site-specific response spectra are used by few consultants, and time history analyses by none. In general the consultants use code equations to determine member capacities, although some also use research results. Secondary lateral force resisting systems are typically used only for structures in which these elements can contribute a significant amount of strength and elastic stiffness. Only half of the consultants check connection capacities; the other half consider only the code force levels, perhaps with a factor of 1.25. None of the consultants estimate connection ductility, although some use their judgment of the connection ductility to estimate a reduction factor for the entire structure. Most of the consultants do not use nonlinear modeling techniques. For frame structures, some of the consultants perform a step-wise linear analysis that follows the formation of plastic hinges. Code drift limitations are typically applied only for frame buildings, although they are usually not a problem for the low-rise buildings typically encountered. Vertical and plan irregularities are generally checked by analyzing the chord and collector stresses and verifying the continuity of the load path. To determine if a building is adequate, most consultants use judgment based on configuration, connection details, calculations, and experience. Experience is usually drawn from each consultant's perception of the performance of buildings in past earthquakes, and the opinions vary dramatically.

Miscellaneous Items. Almost all the consultants have performed damage estimates as part of their evaluations. Most of the consultants indicated they would prefer to provide general descriptions of the type of damage that can occur rather than an actual dollar estimate. Probabilistic methods are not directly applied by the vast majority of the consultants interviewed. All of the consultants report on life-safety hazards caused by nonstructural items. The evaluation procedure typically ends in a written report. The minimum recommended level of strengthening given in these reports is to eliminate the life-safety hazards. Many of the consultants also provide a higher level recommendation that would provide damage control.

The interview and questionnaire results provided a good overall picture of the range of the present state of practice used in the seismic evaluation of existing buildings. In general, a majority of the consultants base their evaluations on a combination of their experience and the local building code. The results of the questionnaire and interview provided information useful in the development of all phases of the ATC-14 methodology.

For more information on the Questionnaire and Interview Process, see Appendix B, which contains the actual questions and a synthesis of all responses. The synthesis of responses includes the most typical answer to each question (summarized above), the range of answers, and any special comments that were given.

2.4 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. This Section will identify and discuss the areas of other research being performed by NCEER investigators which could be useful in performing a seismic evaluation. This will include a discussion of the NCEER projects which could provide information that would improve the ATC-14 document, including the following items:

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.

2.4.1 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 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 (Sections 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 of ATC-14 and Statements 7.1.5.15 through 7.1.5.20 of this report.

2.4.2 Projects Investigating Semi-Rigid Connections in Steel Framed Buildings

Three NCEER sponsored projects are 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 of ATC-14.

2.4.3 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.

2.4.4 Project on Seismic Evaluation of Buildings in New York City

Weidlinger Associates performed 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 are of interest to this project since they 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.

2.4.5 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 to 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 Section 3 of ATC-14. The following Section 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.

2.5 Regional Differences

Another feature of the seismic evaluation of existing buildings which should be developed as a portion of this report are 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.

2.5.1 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 evaluations 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.

2.5.2 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 2.1 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 Ann 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 2.2 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 2.3 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.

2.5.3 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.

2.5.4 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.

2.5.5 Building Code Applications

As 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) 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. In 1991, the BOCA National Building Code (BOCA) mandated the use of seismic design requirements. All municipalities in the southern and southeastern United States requiring the SBC will now be mandating seismic design. With these major advancements, 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 2.6 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.

2.5.6 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.

2.5.7 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.

2.5.8 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 and included into this document.

2.6 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 2.1 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.

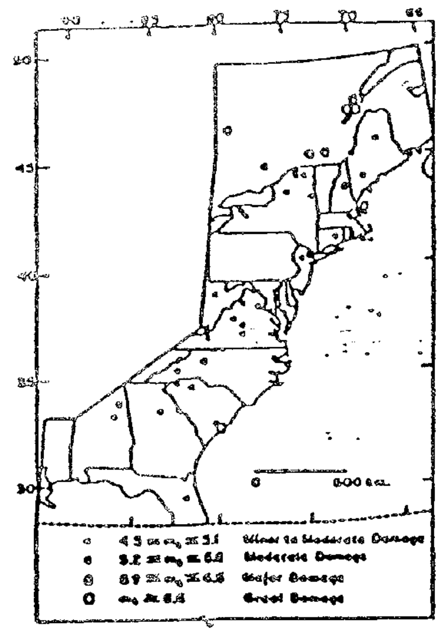


Figure 2.1 - Earthquakes of M_b Equal to or Greater than 4.5 for the Region East of and Including the Appalachians

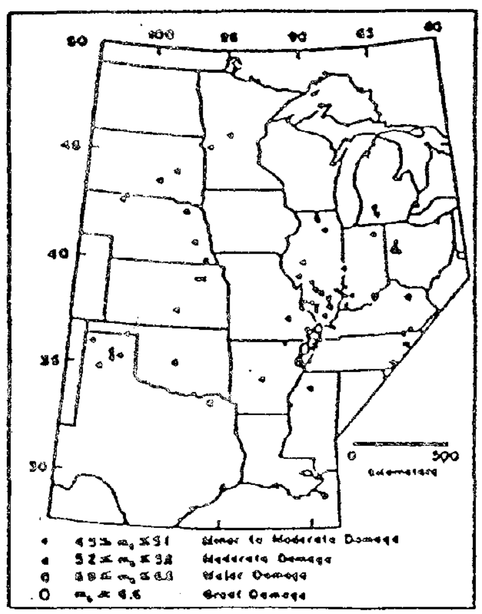


Figure 2.2 - Earthquakes of M_b Equal to or Greater than 4.5 Between the Rocky Mountains and the Appalachians

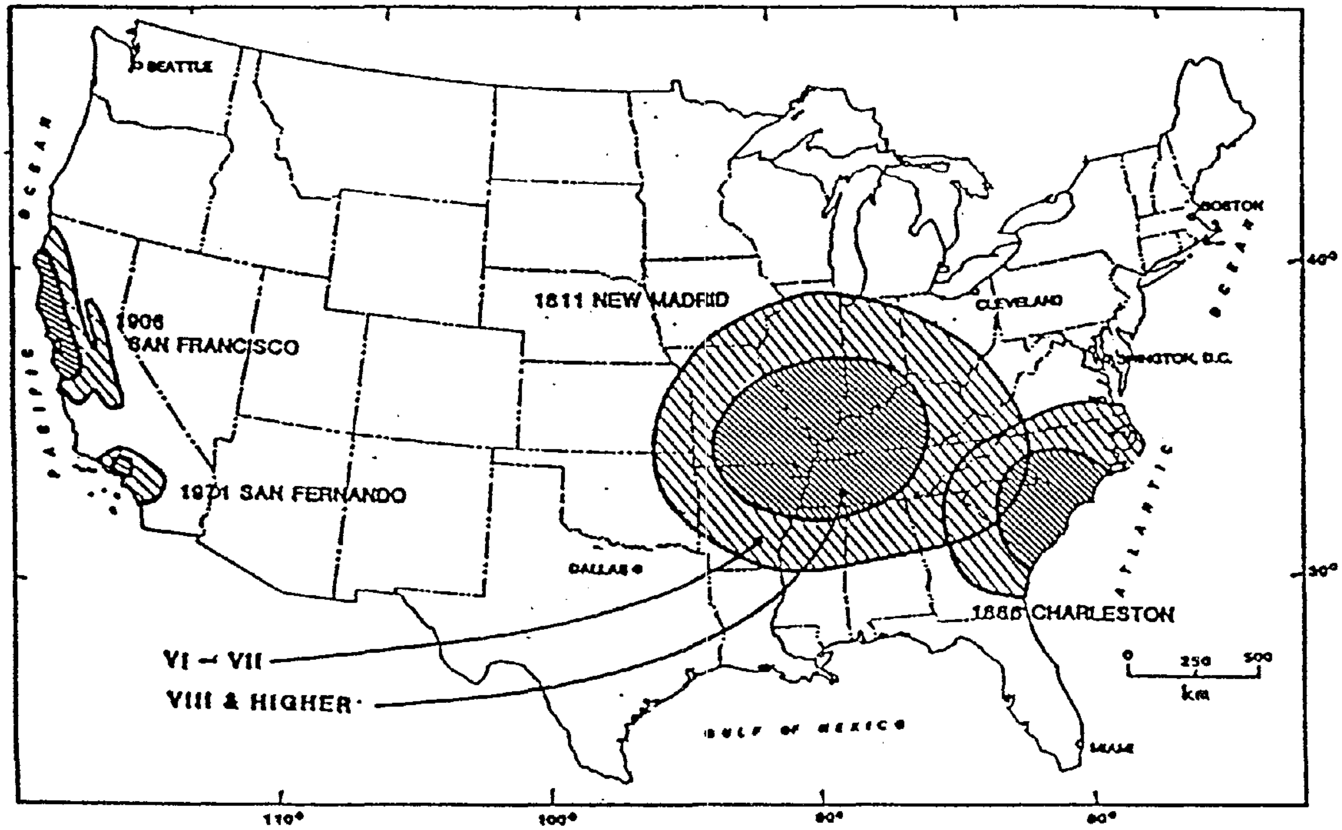


Figure 2.3 - Comparison of Isoseismal Contours for Earthquakes: San Francisco (1906), New Madrid (1811-1812), San Fernando (1971), and Charleston (1886)

TABLE 2.1 - 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
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

Table 2.1 (Continued)

State	Date of Adoption	Mandatory/ Optional	Code
New York	1984	Mandatory Except NYC Uniform Fire Prev. & Bldg.	NY State
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
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
Dist. 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.

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

SECTION 3

SEISMIC LOADING CRITERIA

3.1 Introduction and Discussion of Zoning Maps

Over the last decade, the principles of probabilistic assessment of seismic hazard have become well accepted by both the scientific and engineering communities. There has been active interest in seismic hazard zoning within North America, and several zoning maps have appeared since the release of the original ATC-3-06 report in 1978. A revision of the 1976 Algermissen and Perkins study by Algermissen et al. was published in 1982, and in 1985, Thenhaus et al. published a seismic zoning map of Alaska. Basham et al. (1985) have also presented new strong motion zoning maps of Canada. Included in their presentation are valuable comparative maps showing the different interpretations in adjacent border areas.

The primary difficulty in applying probabilistic seismic hazard in the contiguous states is the problem of attenuation relationships for use in the eastern United States. Basham et al. (1985) summarized the attenuation difficulty by noting that there are very few empirical relationships available for estimating strong ground motion in eastern North America, and there are no strong-motion data for magnitudes greater than 5. Using different relationships it is possible to obtain results that vary by a factor of four for estimates at a single site. Such uncertainty may engender a lack of confidence in any approach, but it should not do so. Seismic zoning mapping is the result of work by concerned professionals who have interpreted the available information to the best of their ability. The end results will show some conservatism, which may be reduced as more data become available. An example of the reduction in conservatism is the recognition that, in the western United States, peak ground acceleration attenuates much more rapidly with distance than previously believed. The result is that some attenuation curves widely used by regulatory groups are recognized as being overly conservative. The use of conservative attenuation relationships is compensated for in some mapping studies by not including the uncertainty of data scatter about the relationship.

The main comparisons of the ATC-3 map of Effective Peak Acceleration (Fig. 3.1) with recent work are with the mapping efforts by Algermissen et al. (1982) and, for areas near the Canadian border, the maps presented by Basham et al. (1985).

Several areas where changes have been made by Algermissen et al. (1982) warrant special comment. Along the central California Coast, the acceleration values have been significantly increased by the inclusion of the source zone extending from Point Conception to offshore San Francisco, which results from the hypothesis that the San Gregorio, San Simeon and Hosgri faults are all connected. This assumption, if accepted, results in the entire coastal zone including Monterey Bay and Santa Barbara being included inside the 0.4 g contour. The reasons for the change in the zoning of the northern California coast area are not as clear, but these suggest that all of the California coast with the exception of San Diego should be included within a 0.4 g contour.

In the Seattle area, the new Algermissen et al. (1982) results agree more closely with the A_a map (Fig. 3.1) than the earlier study by Algermissen and Perkins. There has been considerable discussion recently (Heaton and Hartzell, 1985) about the possibility that a major subduction zone may lie beneath western Washington State. This may raise estimates of the size of the largest earthquake in the region, but would not be expected to influence the probabilistic zoning to a large extent.

In the southeastern United States, the extent of the region capable of producing an event like the 1886 Charleston earthquake has been an ongoing controversy as the geologic or tectonic features associated with the Charleston event have not been described. This was the reason for the extensive 0.1 g contour on the A_a map that covered a portion of Georgia, the Carolinas, Tennessee, and Virginia. Recent discovery of earlier sandblown artifacts in the Charleston area suggests that future studies might be able to reduce the size of this zone.

In New England, the revised Algermissen et al. (1982) study reduces the extent of the 0.1 g contour. With the recent events in New Hampshire and New Brunswick, the ATC-3-06 map, which has closer agreement with Basham et al. (1985), is preferred. The extended 0.1 g contour presented with the ATC-3-06 document is in good agreement at the Canadian border with the more recent work by Basham et al., which includes the influence of these more recent events.

It is recommended that the A_a map of Effective Peak Acceleration, and the A_v map of Effective Peak Velocity-Related Acceleration, reproduced here as Figures 3.1 and 3.2, respectively, be used for this study. With the possible exception of the central California coast area, the maps may be used without change. Including the Santa Barbara to Monterey coastal area within the 0.4 g contour is in agreement with the most recent recommendations of the Structural Engineers Association of California (SEAOC, 1985). It is recommended that values of A_a and A_v for sites located between contours be determined through linear interpolation.

3.2 Response Spectra

When designing a new structure, it is usually possible to add a measure of conservatism to the design at little extra cost. This conservatism is applied by increasing the building code seismic loading function proportionately for longer period structures. For the review of existing buildings, the primary aim is to evaluate the true strength of the structure when subjected to an earthquake. Although significant scatter exists among earthquake data, the aim of this study should be toward the evaluation of the effect of the most probable large event to which the structure would be subjected within a specified lifetime. This loading can be represented by the mean response spectra appropriate for the particular motion level and the soil profile beneath the structure.

Response spectra were not included as part of the provisions section of the ATC-3-06 document, but were included without detailed description in the commentary. The response spectra shown in the commentary have been reconstructed and are presented here as Figure 3.3. The key ground motion parameters (velocity/acceleration (v/a) and dimensionless parameter values (ad/v^2) to be used with the spectral amplification table values to develop the ATC-3-06 spectra are provided in Table 3.1. The ATC-3-06 spectra are similar to those that would be obtained by using the 84th percentile spectral amplification values given by Newmark and Hall (1982).

It is possible to use the 50th percentile amplification parameters (see Table 3.2, which is taken from Newmark and Hall (1982)) to compute the median spectra equivalent to those given in the ATC-3-06 commentary. Median response spectra for 5 percent damping in the region with A_a of 0.4 g using this procedure, are shown in Figure 3.4. The spectral values in Figure 3.4 represent the highest values recommended for any zone in this study. The reduction of A_a from 0.4 to 0.3 for soft soil conditions is not recommended for this study. This reduction has not been well supported by recent strong motion data (Joyner and Boore, 1981). As existing buildings in locations where soil profile type 3 exists may also have unknown residual stress from foundation deformation, the additional refinement of reducing the A_a value is not warranted. In areas where the A_v and A_a values are not equal, the spectral construction is somewhat different. Where the A_v value is greater than A_a, the velocity value used with the Newmark and Hall amplification factors should be computed using the A_v value. The zero period acceleration value and the acceleration value used with the acceleration amplification factor is the A_a value. The spectral construction procedure, based on the procedure proposed by Newmark and Hall (1982), is described in more detail in Section 3.5. But, as an example, consider the recommended response spectra for soil sites in two alternative locations - Sacramento, California and Memphis, Tennessee. The city of Sacramento has mapped values of 0.2 and 0.3 for A_a and A_v, respectively, whereas the city of Memphis has values of 0.2 for both components. The spectra for these two cities constructed in accordance with the procedures described in Section 3.5 are shown in Figure 3.5.

A separate suggestion that the median response spectra is appropriate, and not unconservative, is supported by a comparison with some probabilistic response spectra recently developed for sites in Southern California. Using a seismic source model together with response spectra acceleration attenuation equations, it is possible to develop an entirely risk-based response spectra for a specific site. At present, these studies must be considered tentative, as they are available only for a few tectonic regimes. Preliminary results are shown in Figure 3.6, where the mean response spectra for several different sites is compared with the response spectrum recommended in this study for the same soil profile type. The risk-based response spectrum has an estimated recurrence interval of 500 years, with approximately a 90 percent probability of not being exceeded in 50 years. The values in the risk-based spectra have been changed so that the zero period acceleration is the same for both curves given in Figure 3.6.

It should be stressed that the maps in Figures 3.1 and 3.2, and the response spectra recommendations for use with them have been developed for use as a single consistent package. If, for example, other maps, response spectra, or other suggested revisions to the guidelines in this Section become available (subsequent to publication of this report), they should not be used with the remaining portions of these guidelines until their overall effect and consistency have been carefully examined.

3.3 Response Spectra for Different Damping Levels

Response spectra for different damping levels can be constructed using the same procedures as were used for the 5 percent damping level spectra shown on Figure 3.4 by selecting the alternate appropriate damping levels from Table 3.2. The spectral construction method is the same as recommended by Newmark and Hall, and is outlined in detail in Section 3.5.

3.4 Ground Motion and Response Spectra for Differing Probability Levels

Some interest has been expressed by the Project Engineering Panel in possibly modifying both the ground-motion level and the response spectral shape to represent the effect of the shorter assumed life of a strengthened structure compared to a new building. The direct effect of the shorter assumed life would be a reduction of the A_a and A_v values below those shown in Figures 3.1 and 3.2. The response spectral shape will also change with the assumption of a decreasing lifetime. The major contribution to a probabilistic response spectra in the period range controlled by the ground velocity comes from high-magnitude events. As large events have longer recurrence intervals than smaller events, their contribution to the response spectra in the velocity controlled range will fall faster than the reduction in the maximum spectral acceleration. This effect is demonstrated in Figure 3.7, where the probabilistic response spectra for range of mean recurrence intervals for a rock site are shown. The changing spectral shape and the more rapid decrease of the spectral values beyond a 0.3 second period as the recurrence interval is reduced are readily apparent.

The response spectra construction procedure recommended here is based upon the use of a zero period acceleration value, which is used to scale the overall response spectra, and a velocity to acceleration ratio, which is used to determine the relative shape of the response spectra. The effects of decreasing the assumed lifetime of these parameters have been evaluated from a few specific site studies. The change in the values of these parameters may be expected to vary in different seismic regimes, just as the contour values in Figures 3.1 and 3.2 change. However, for the purposes of this study, it is believed that average values of the parameters obtained from a range of studies would be appropriate.

From the results of various seismic hazard analyses in several areas of the United States and its territories, it is possible to plot the peak acceleration values against the estimated recurrence interval in years (Fig 3.8). In this figure, the numerical values have been changed so that all curves cross at 475 years, the recurrence interval equivalent to the 10 percent probability of exceedance in 50 years. The data shown on Figure 3.8 give an approximate estimate of the range of variation. Examination of the data in Figure 3.8 indicates that the reduction is somewhat less where deep earthquakes occur. Over the range of greatest interest between 100 and 500 years, the differences are small, so the use of the mean relationship shown on Figure 3.8 is the recommended approach. From the mean relationship, a simple scaling relationship has been derived, which can be used where different probabilities than those upon which the maps are based are required. This relationship is:

$$F = 0.18\, T_1^{0.28} \qquad (3.1)$$

where F is the multiplication factor for the map acceleration values for different return periods T_1. The line represented by Equation 3.1 is the recommended curve shown on Figure 3.8. The value of T_1 in years, appropriate for different probability levels, can be obtained by assuming the Poisson distribution as follows. As an example, for a 20 percent probability of exceedance in 50 years $T_1 = (-50) / \ln (1 - 0.2)$, or 224.1 years. Use of 224 years in Equation 3.1 will give a value of 0.82 for F. The A_a and A_v values should both be scaled by this factor when the response spectrum for the reduced probability is required.

Figure 3.7 shows that the velocity to acceleration (v/a) ratio changes with the recurrence interval. The spectra shown on Figure 3.7 are for a rock site. Similar spectra have been developed probabilistically for silts with soil profiles and the overall variation in v/a examined. From this work, a relationship for modification of the v/a ratio of 24, 36 and 48 inches per sec per g used with 10 percent probability of exceedance in 50 years, or T_1 of 475 years, have been developed. This relationship gives a scaling factor as follows:

$$F_{va} = -0.02 + 0.38 \log T_1 \qquad (3.2)$$

where T_1 is the recurrence interval for the changed probability as described above. The data points used to derive Equation 3.2, and the variation of v/a represented by the equation, are shown on Figure 3.9. For the 20 percent exceedance in 50 year example, the factor is 0.88 and the v/a ratio for spectral development of the soil site would be 0.88(36) or 31.7 in/sec/g. The comparative response spectra for a stiff soil site with mapped A_a and A_v values of 0.4 g are shown on Figure 3.10 for three different probability levels. Ten percent probability of exceedance in 50 years represents the basic criterion from these recommendations and produces the highest of the three spectra shown on Figure 3.10. The two lower spectra represent reduced exceedance probability levels, which may be used if consideration is given to the probably shorter lifetimes of existing buildings. Reduced probabilities could provide a means for local ordinances allowing a lesser level of compliance for non-critical structures in exchange for an agreed upon future demolition date. Such a requirement, however, would be a one time only arrangement that could not be renewed.

3.4.1 Duration

The duration of the strong shaking, as well as the strength of the shaking, becomes important in reviewing existing buildings with limited ductility. With limited ductility, there may be little reserve strength to withstand more than one or two excursions beyond the yield strength. The duration appropriate for these guidelines is not well defined and perhaps should be considered a constant value for all seismic zones in which the A_v value is 0.10 or greater. The duration of strong shaking increases with the magnitude of the earthquake. While the maximum ground motion levels attenuate with distance, the duration during which the peak values will be close to the attenuated value increases. These two factors change the duration with opposite and compensating effects so that, when probabilistic ground motions are considered, the effective duration is not a function of the ground motion level. These aspects of duration were discussed in detail by McGuire and Barnhard (1979).

Considering the values used for some major projects where durations for a range of conditions were required, a reasonable value is believed to be in the range of 15 to 20 seconds. It should be recognized, however, that when a deterministic estimate of the extreme level event close to a major fault is considered, the duration of that event will be a function of the magnitude, and may be considerably longer than 20 seconds.

3.5 Recommended Response Spectra Development Procedures and Example Computations

The procedures for developing "standard" 5 percent damped response spectra and for developing spectra for various levels of damping and probability are based on the procedures proposed by Newmark and Hall (1982) as well as the methodology described above. Following are two illustrative examples:

3.5.1 Example 1: Development of Standard 5 Percent Damped Spectra

For example purposes, we have chosen a hypothetical site in Salt Lake City, Utah. The first step in the development of the design review conditions and the standard 5 percent response spectra is to use the contour maps on Figures 3.1 and 3.2 to obtain the values of A_a and A_v, respectively. These give values of 0.2 for both components. The site soil profile selected for this example is Soil Profile S2 (deep stiff soil), which is the most representative profile in the Salt Lake City area.

As indicated previously, the standard 5 percent damped response spectra is defined as that comparable to the ATC-3-06 spectra having a 10 percent probability of exceedance in 50 years. The v/a ratio and the ad/v^2 term appropriate for the S2 profile, taken from Table 3.1, are 36 in/sec/g and 4, respectively. From these values, we can estimate the maximum ground motion parameters as follows: $a_{max} = A_a = 0.2\ g$

$$v_{max} = A_v \times (v/a) = 0.2 \times 36 = 7.2 \text{ in/sec}$$

$$d_{max} = \frac{(ad/v^2)\ (v^{max})^2}{a_{max}\ (32.2 \text{ ft/sec}^2/g)\ (12 \text{ in/sec})} = \frac{4 \times 7.2 \times 7.2}{0.2 \times 32.2 \times 12} = 2.7 \text{ in}$$

The maximum spectral values can be obtained from the ground motion values above by using the amplification values given in Table 3.2 for 5 percent damping. These computations are shown in Table 3.3.

Using the methods prescribed by Newmark and Hall (1982) we can now draw the desired response spectra on tripartite log paper. The procedure, outlined in Table 3.4 and illustrated in Figure 3.11, is as follows:

1. Complete the short-period end of the spectrum by drawing a constant acceleration-response line ($S_a = A_a$) for all periods less than 0.033 second. This segment is the line between points A and B, Figure 3.11.

2. Complete the segment of the spectrum between periods of 0.033 sec and 0.125 sec. S_a equals A_a at T = 0.033 sec and S_a equals A_a at T = 0.033 sec and S_a(max) at T = 0.125 sec. This segment is shown in Figure 3.11 as the line between points B and C.

3. Complete the segment of the spectrum between the period of 0.125 second and the period at which lines of constant S_a(max) and $S_{v(max)}$ intersect. This segment, shown on Figure 3.11 as the line between points C and D, is a constant acceleration-response line ($S_a = S_a$(max)). The equation used to calculate T at the intersection of lines of constant S_a(max) and S_v(max) is provided in Table 3.4 (point D). In this case, T = 0.456 sec.

4. Complete the segment of the spectrum between the period at which lines of constant S_a(max) and S_v(max) intersect and the period at which the lines of constant S_v(max) and S_d(max) intersect. This segment, shown on Figure 3.11 as the line between points D and E, is a constant velocity-response line ($S_v = S_v$(max)). The equation used to calculate the value of T at the intersection of lines of constant S_v(max) and S_d(max) is provided in Table 3.4 (point E). In this case, T = 1.962 sec.

5. Complete the long-period segment of the spectrum by drawing a constant displacement-response line ($S_d = S_d(max)$) between the period at which lines of constant $S_v(max)$ and $S_d(max)$ intersect and a period of 4.0 sec. This segment is the line between points E and F, Figure 3.11.

By the use of the relationships given in Table 3.4, it is also possible to draw the response spectra on an arithmetical acceleration response scale plot. Figure 3.12 shows such a plot, including points A through F, for this example case.

3.5.2 Example 2: Development of Response Spectra with Probabilities and Damping Different from the Standard Spectra

Example 1 illustrates how to develop standard response spectra having a 5 percent damping level and 10 percent probability of being exceeded in 50 years (computations are provided in Tables 3.3 and 3.4). If damping level and/or probability criteria are different from that for the standard spectra, the equations provided in Table 3.4 are still valid, but the maximum spectral values developed in Table 3.3 will be different. As an example to show the effect of different damping and probability criteria, assume the following site conditions and criteria:

Site:	Las Vegas, California
Profile:	Deep stiff soil
Probability:	30 percent probability of exceedance in 50 years
Damping Level:	10 percent of critical

Using the contour maps of Figures 3.1 and 3.2 and linear interpolation, we obtain values of 0.07 and 0.15 for A_a and A_v, respectively. These values must first be changed to represent the different probability levels. This is done by using Equation 3.1. Thirty percent probability in 50 years corresponds to a mean recurrence interval T_1 of 141 years. The reduction factor F given by Equation 3.1 is 0.72, so the A_a and A_v values used for spectra construction should be 0.05 and 0.11, respectively. The v/a ratio should also be reduced for the different probability level using Equation 3.2. Equation 3.2 gives a reduction factor of 0.8, which results in a v/a ratio for the deep still soil (S2) profile of 28.6 in/sec/g. Using these values we obtain the maximum spectra values shown in Table 3.5.

The values from Table 3.5 can now be used directly with the equations given in Table 3.4 to obtain both the period values, where the straight line segments on the tripartite spectrum intersect, and the spectral acceleration value for 10 percent damping at any intermediate period value of interest. For example, at a period of 1 second, the spectra value is given by $S_a = (2\pi/T)$ $S_v(max)$, or $2\pi \times 4.4 = 27.6$ ins/sec^2, or 0.07 g. The complete response spectrum for this example is shown on Figure 3.13. The effect of the different parameters on the overall spectral shape is readily apparent by comparison with the standard curve shown on Figure 3.12.

3.6 SEISMICITY ISSUES

During their review of this Section of the ATC-14 document, the NCEER 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.

3.6.1 Applicability of the Seismic Zoning Maps

During the initial meeting of their project, the NCEER 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

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.

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 Section 3 of ATC-14. Lamont-Doherty will begin work on these issues.

6. As presented, Section 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 Section 3 of ATC-14 should be published. This documentation could be in the form of a technical paper.

3.6.2.4 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 3.15 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 3.16 and 3.17 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 3.16 shows the settlement of soil about 16 inches as seen from a school building which is on piles. Figure 3.17 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 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.

3.6.2.5 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 3.18, 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, in some cases 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.

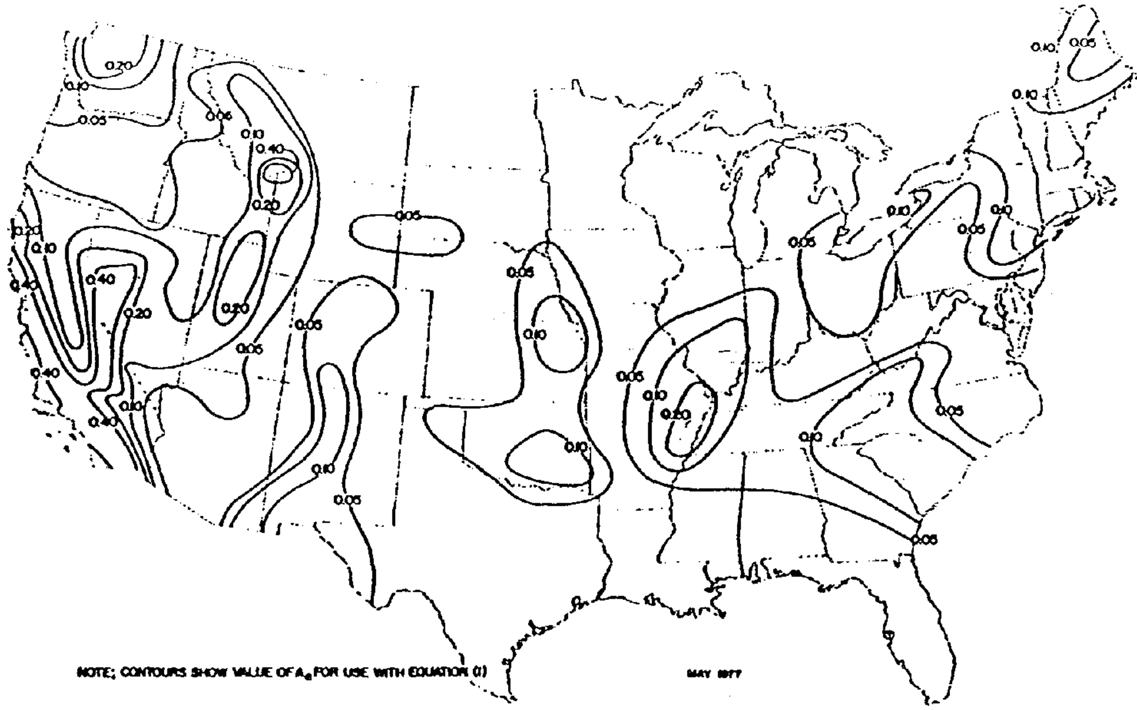
The third reason for concern stems from past disregard. As observed by Stratta (1987), despite its major damage potential, liquefaction has not been a major concern in design in the United States. Relatively few existing buildings in the United States have been constructed to resist earthquakes. And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

3.6.2.6 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.

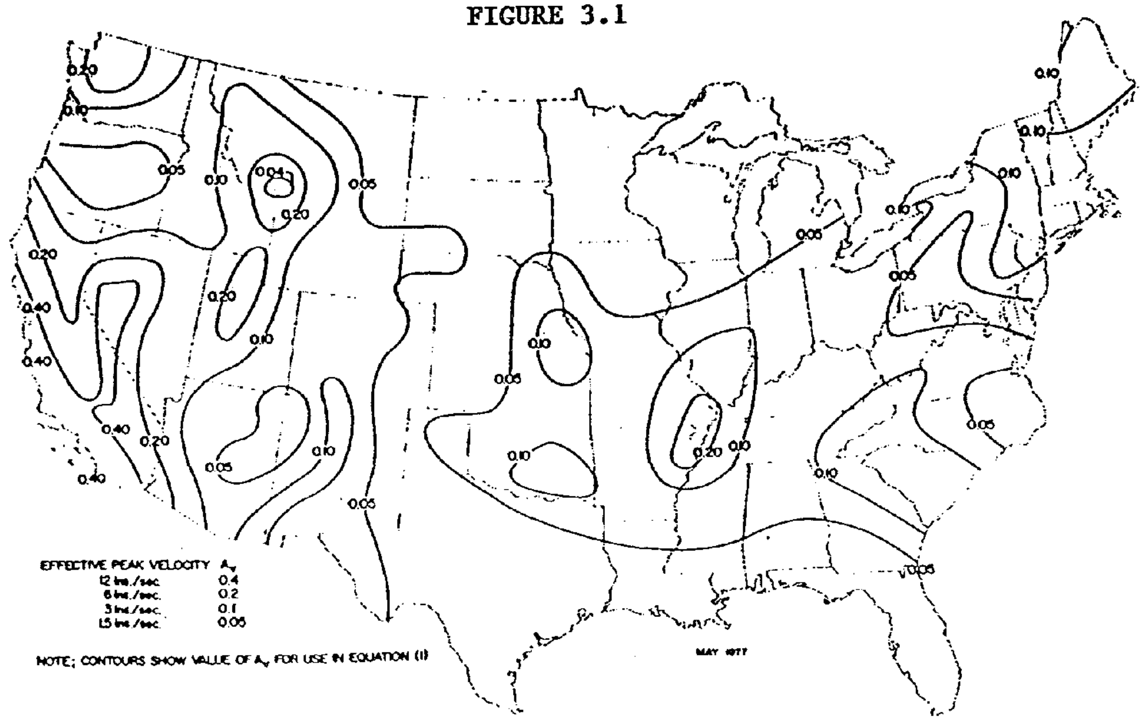
Because of the wide range in possible foundation options in existing buildings, the ATC-14 methodology should recommend that the liquefaction potential at a site be assessed and, if found to be positive, that the technical problem be referred to a qualified geotechnical engineer for resolution. Liquefaction 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 D. 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).



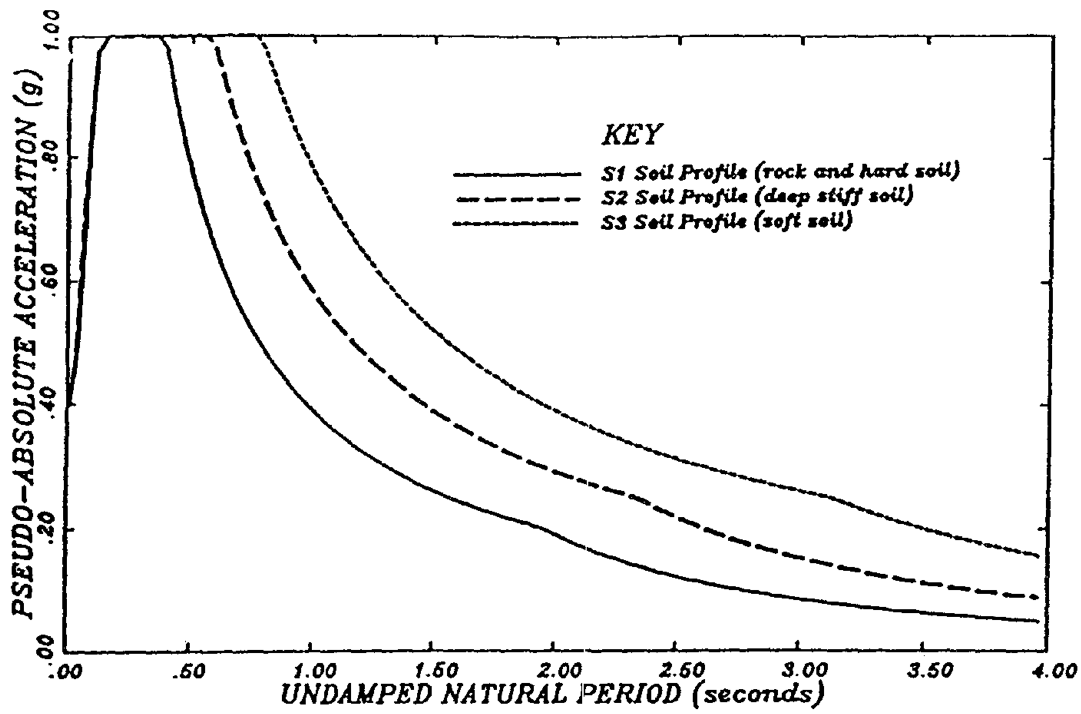
Contour Map of Effective Peak Acceleration (EPA) for the 48 Contiguous States. Contours Represent EPA Levels with a Non-Exceedance Probability of between 80 and 95 Percent during a 50-Year Period. Larger Scale Version of This Map Showing County

FIGURE 3.1



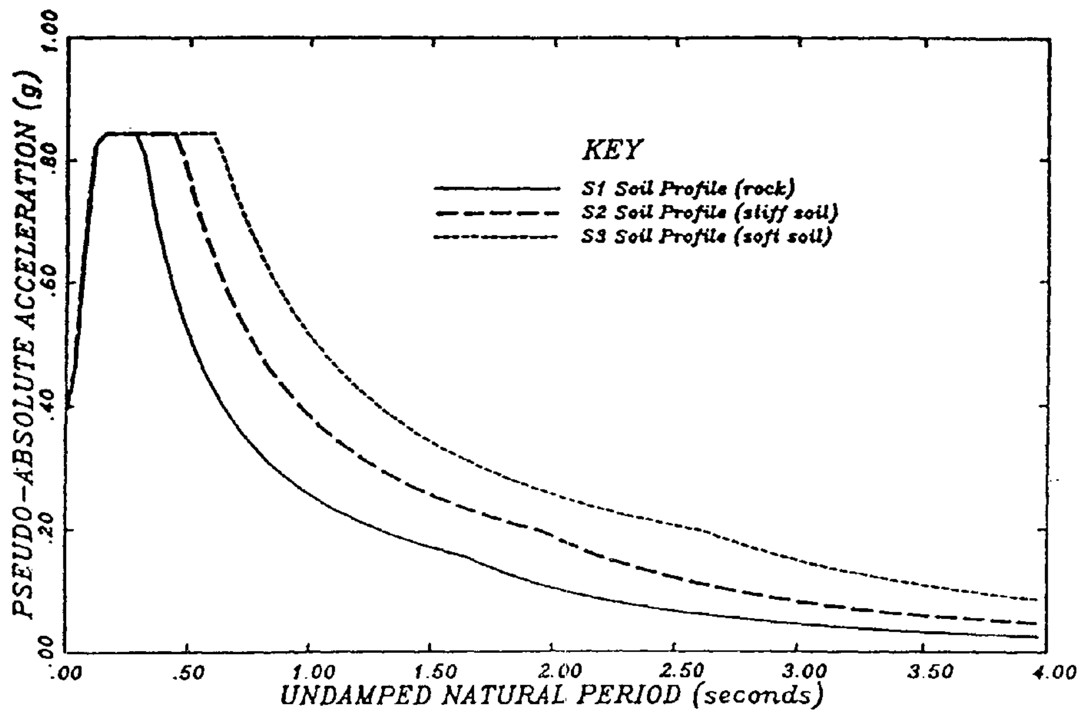
Contour Map of Effective Peak Velocity-Related Acceleration (EPV) for the 48 Contiguous States. Contours Represent Values Based on Velocity But Equivalent to Acceleration for Use in Developing Lateral Design Forces. Larger Scale Version of

FIGURE 3.2



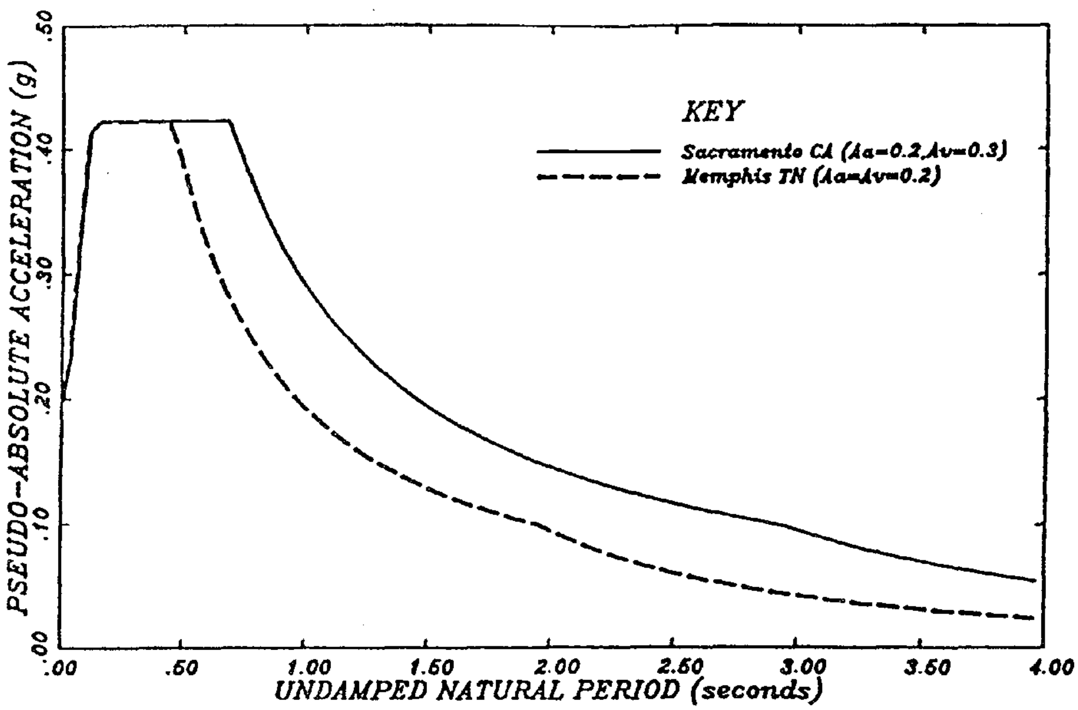
Response Spectra Given in Commentary of ATC-3-06 Document (Damping = 5%).

FIGURE 3.3



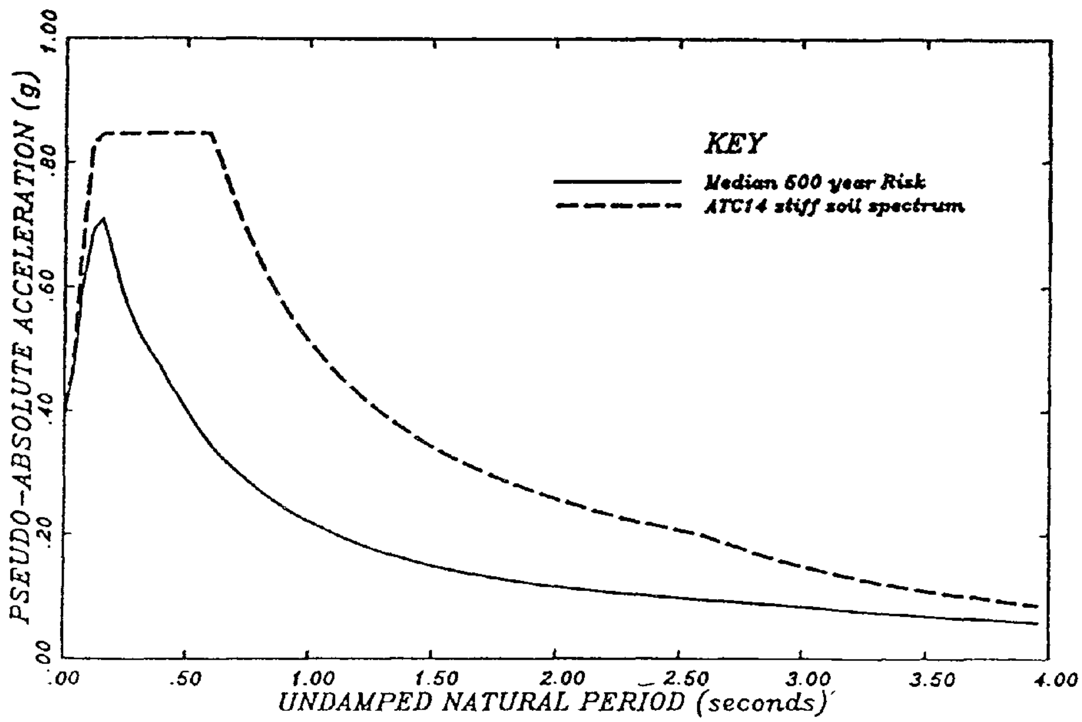
Recommended Response Spectra to be Used for Existing Buildings (A_a = .4; Damping = 5%).

FIGURE 3.4



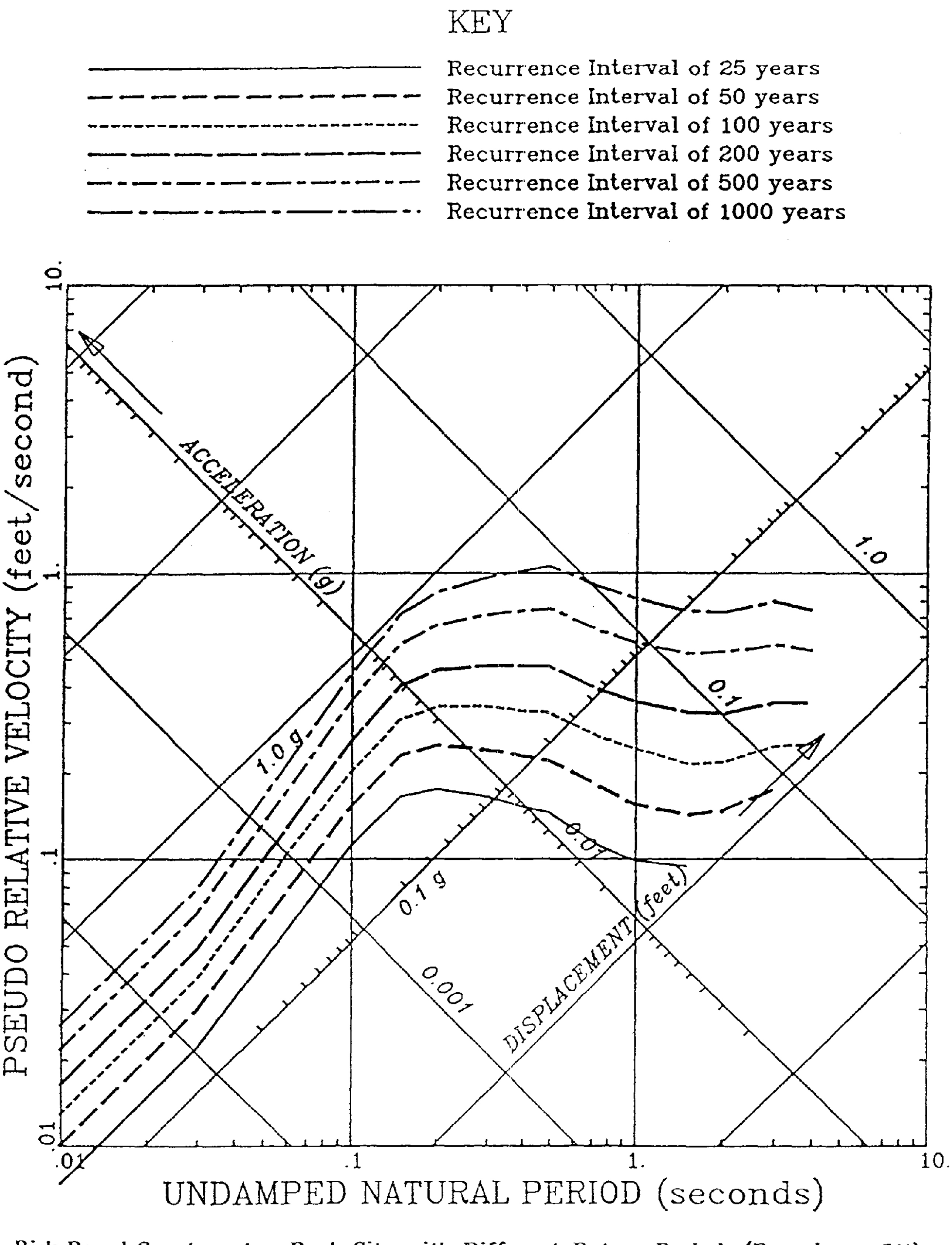
Comparison of Spectra for Two Cities with Differing A_a and A_v Values (Damping = 5%).

FIGURE 3.5



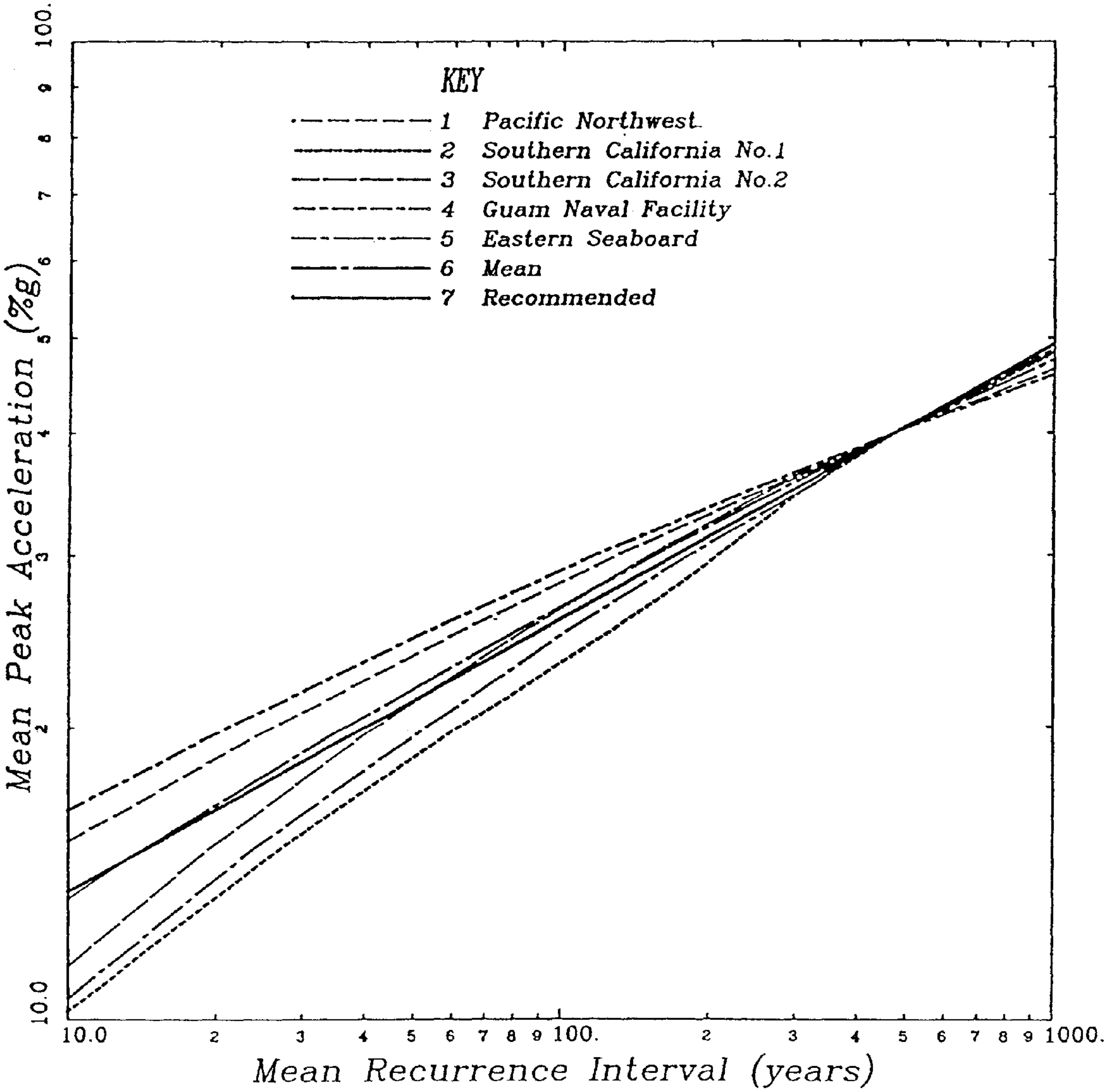
Comparison of Recommended ATC-14 Soil Spectra with Risk Based Spectra (Damping = 5%).

FIGURE 3.6



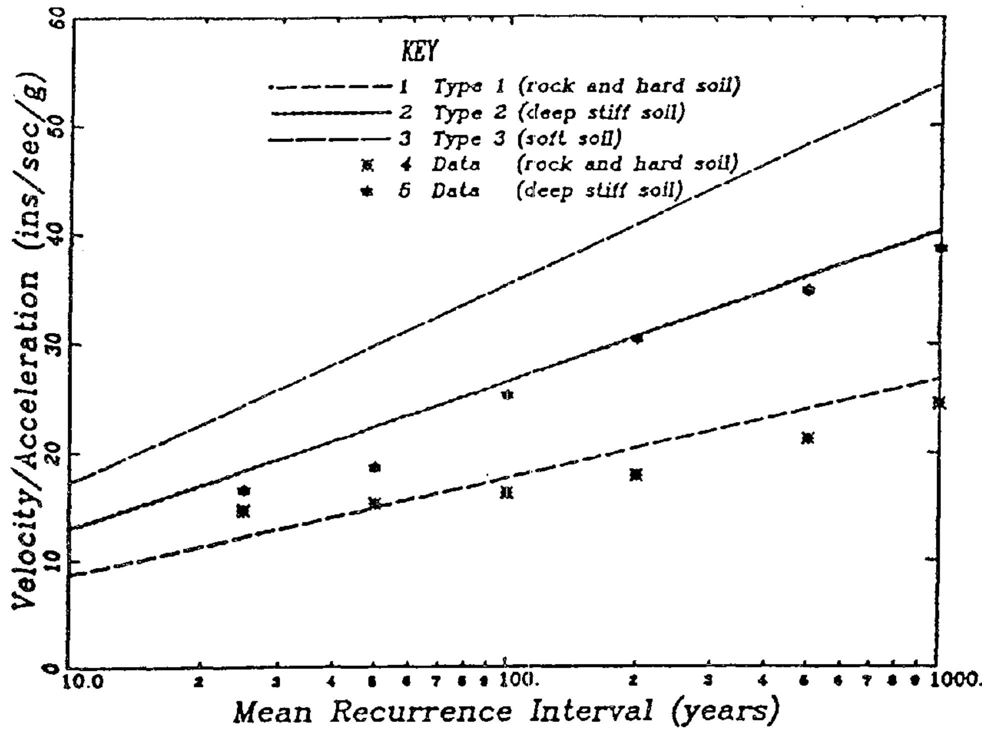
Risk-Based Spectra at a Rock Site with Different Return Periods (Damping = 5%).

FIGURE 3.7



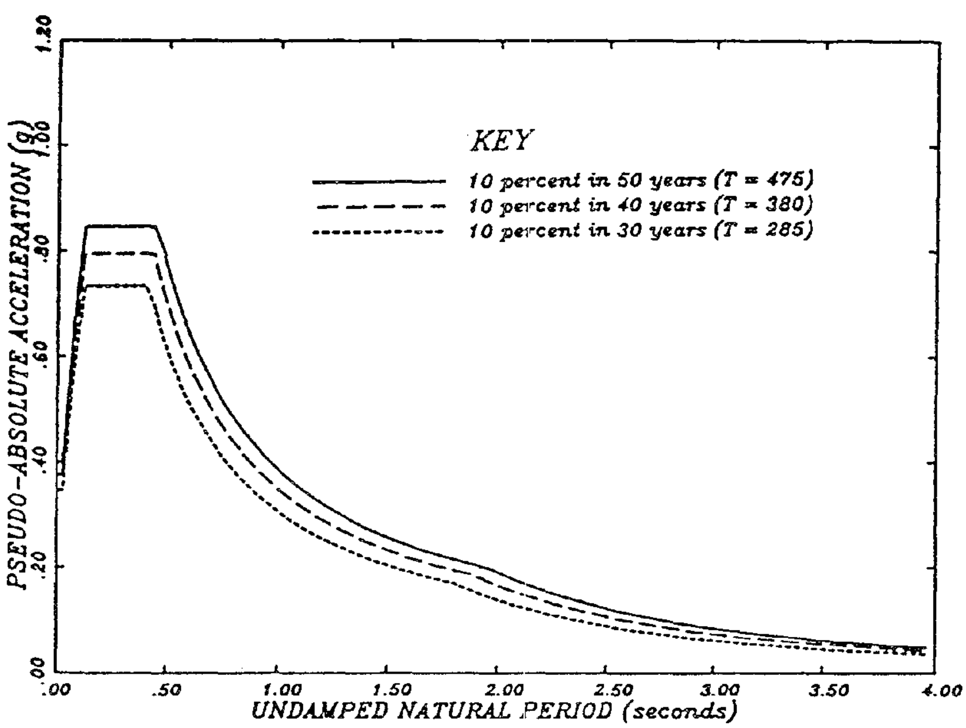
Examples of Recurrence Interval Variations Used to Obtain the Recommended Equation. All Curves Scaled To Give the Same Value at 475 Years.

FIGURE 3.8



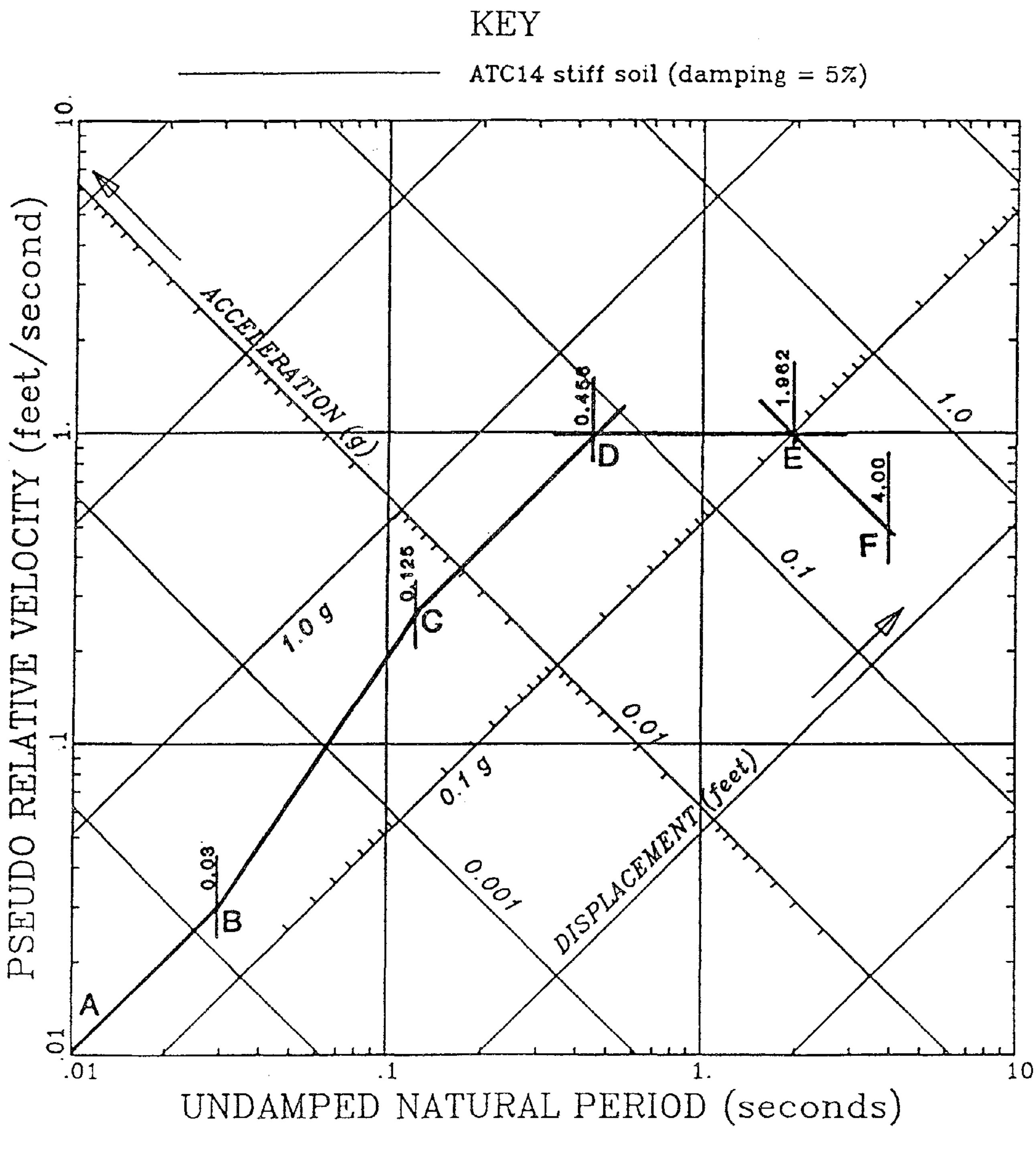
Recommended Variation of (v/a) Ratio for Changing Recurrence Intervals. Values at 475 Years Are Consistent with Basic Guidelines.

FIGURE 3.9



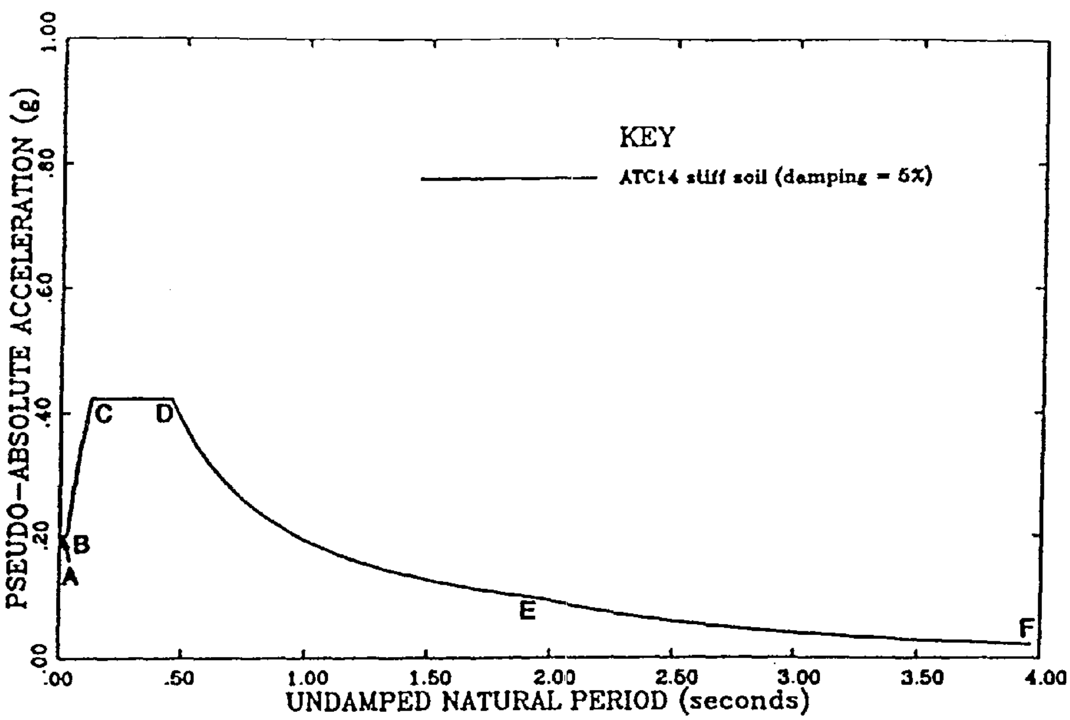
Comparison of 0.4 g Contour Spectra for Differing Probability Levels (Damping = 5%).

FIGURE 3.10



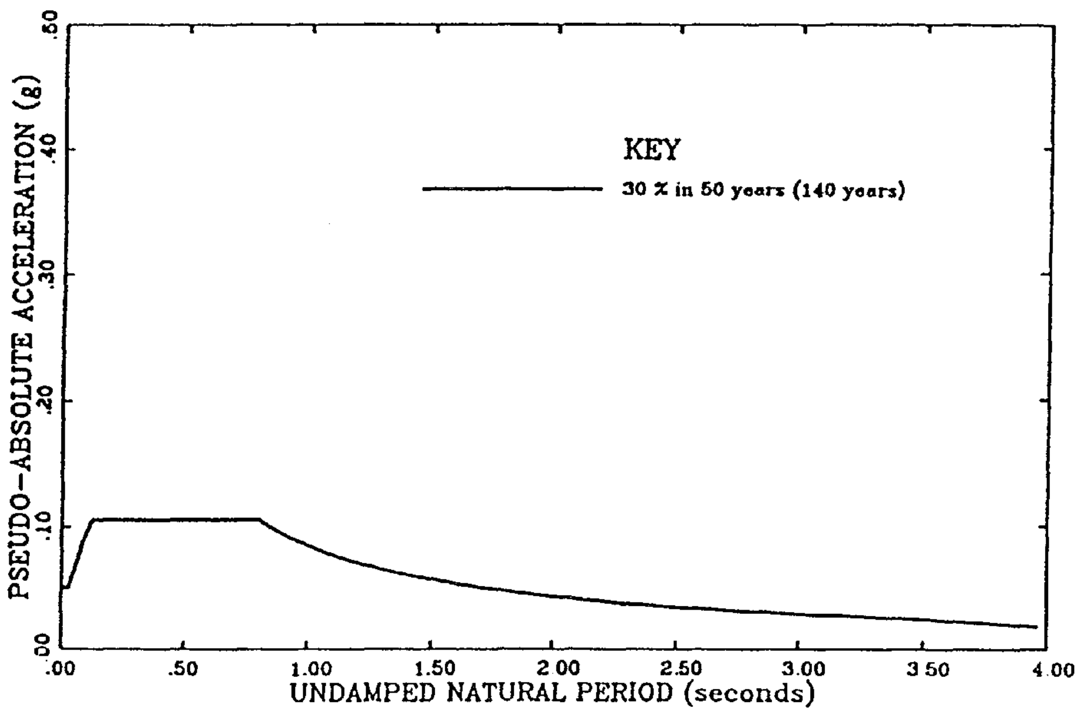
Demonstration of Spectral Construction on Tripartite Log Scales (Damping = 5%).

FIGURE 3.11



Arithmetical Acceleration Scale Presentation of Figure 3.11 Data (Damping = 5%).

FIGURE 3.12



Las Vegas Soil Site with Reduced Probability and Increased Damping (Damping = 10%).

FIGURE 3.13

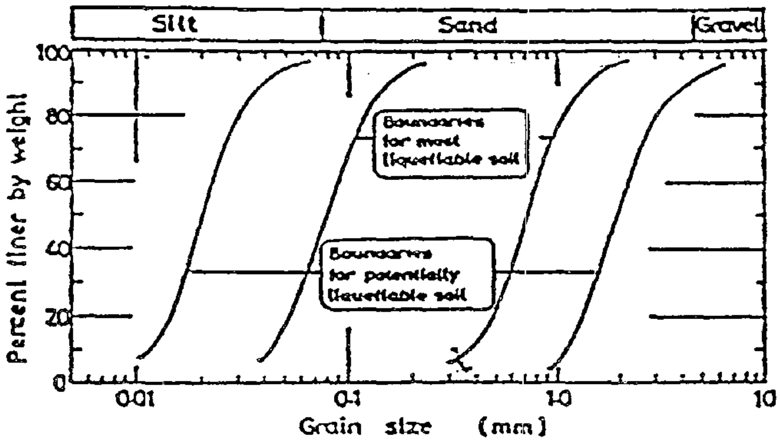


Figure 3.14 - Soil Potential for Liquefaction
(Ishihara, 1985)

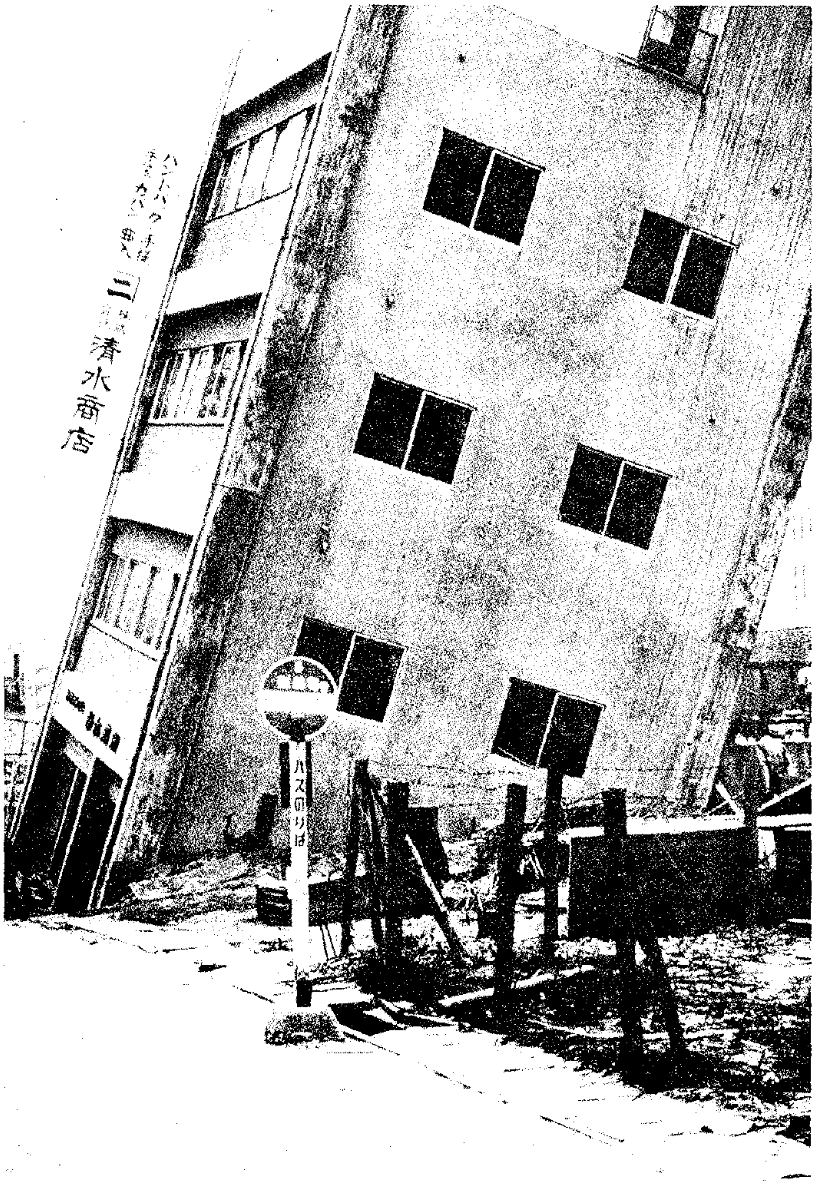


Figure 3.15 - Building Damaged by Soil Liquefaction
(Seed and Idriss, 1982)

Figure 3.16- Soil Settlement Due to Liquefaction
(Bertero, et. al.,1985)

Figure 3.17 - Slab on Grade Settlement Due to Liquefaction
(Bertero, et. al., 1985)

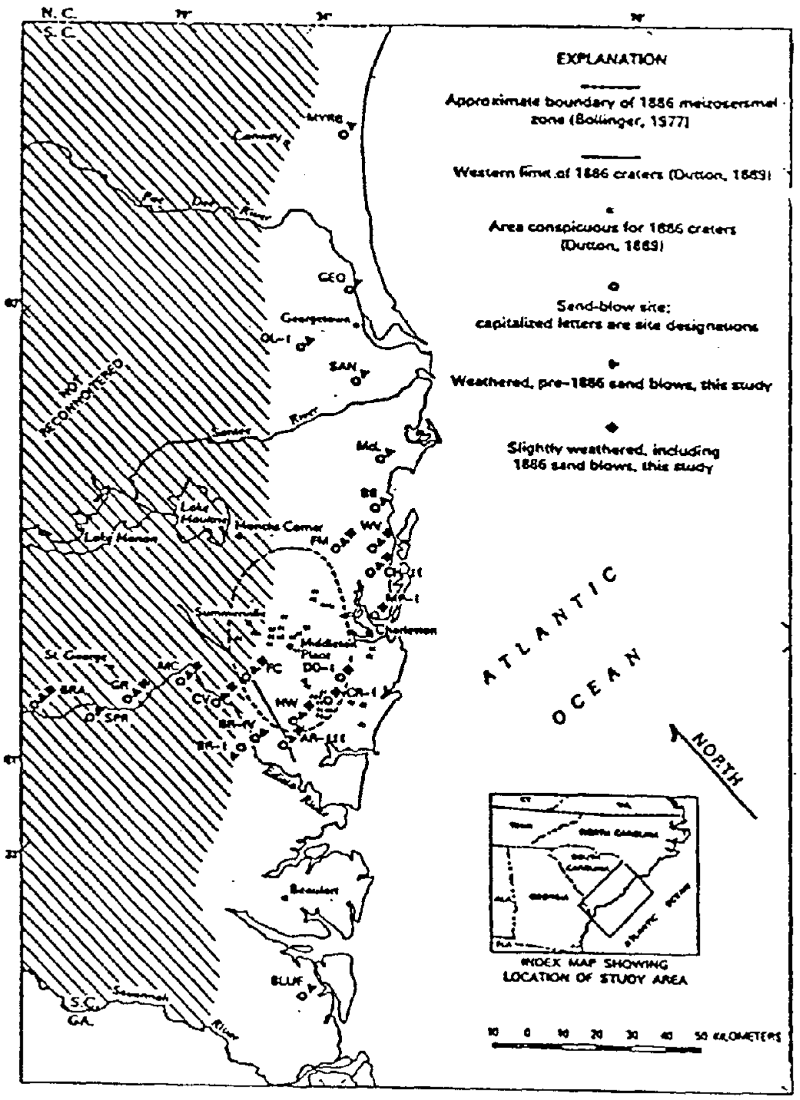


Figure 3.18- Map of 1886 and Pre-1886 Sand Blow Sites in South Carolina (Gohn, et. al., 1984)

TABLE 3.1

Key Ground Motion Parameters To Be Used with Spectrum Amplification Factors for Horizontal Elastic Response

	Soil Profile	v/a (in/sec/g)	ad/v^2 (Dimensionless)
1	(Rock and Hard Soil)	24	5
2	(Deep Stiff Soil)	36	4
3	(Soft Soil)	48	4

TABLE 3.2

Spectrum Amplification Factors for Horizontal Elastic Response (Newmark and Hall, 1982)

Damping (Percent Critical)	Median (50%)		
	A	*V*	*D*
0.5	3.68	2.59	2.01
1	3.21	2.31	1.82
2	2.74	2.03	1.63
3	2.46	1.86	1.52
5	2.12	1.65	1.39
7	1.89	1.51	1.29
10	1.64	1.37	1.20
20	1.17	1.08	1.01

or, if equations are preferred:

For Acceleration: Factor = $3.21 - 0.68 \ln \beta$
For Velocity: Factor = $2.31 - 0.41 \ln \beta$
For Displacement: Factor = $1.81 - 0.27 \ln \beta$

β = Percent critical damping.

TABLE 3.3

Maximum Spectral Values for Hypothetical Salt Lake City Site

$S_a(\max)$	$= 0.2 \times 2.12$	$= 0.42$ g
$S_v(\max)$	$= 7.2 \times 1.65$	$= 11.9$ in/sec
$S_d(\max)$	$= 2.7 \times 1.39$	$= 3.8$ in.

TABLE 3.4

Computation of Spectral Values and Tripartite Intersections for Hypothetical Salt Lake City Site

Point	Period (secs)	Acceleration S_a (g)	Equations for T* and S_a**
A	0.01	0.2	T is fixed S_a = constant = A_a
B	0.03	0.2	T is fixed $\log(S_a) = \log(A_a) + 2.46\log(\text{factor}^{***}) + 1.61\log(\text{factor})\log(T)$
C	0.125	0.423	T is fixed S_a = constant = $S_a(\max)$
D	0.456	0.423	$T = 2\pi\ S_v(\max) / S_a(\max)$ $S_a = (2\pi/T)\ S_v(\max)$
E	1.962	0.0982	$T = 2\pi\ S_d(\max) / S_v(\max)$ $S_a = (2\pi/T)^2\ S_d(\max)$
F	4.0	0.0234	T is fixed

*T is the undamped natural period in seconds

**S_a is the response spectral acceleration (correct fps units must be used in the above equations)

***factor is the acceleration amplification factor for the chosen damping level given in Table 3.2 (2.12 for the Fig. 3.11 example)

TABLE 3.5

Maximum Spectral Values for Hypothetical Las Vegas Site
(Damping = 10%)

	Ground Values	Spectral Maxima
Acceleration	**0.05 g**	**0.05 x 1.64 = 0.08 g**
Velocity	**0.11 x 28.4 = 3.2 in/sec**	**3.2 x 1.37 = 4.4 in/sec**
Displacement	**2.05 in**	**2.05 x 1.2 = 2.5 in**

SECTION 4

GENERAL METHODOLOGY FOR THE EVALUATION OF EXISTING BUILDINGS

This Section presents the methodology recommended for the seismic evaluation of existing buildings. The methodology was developed on the basis of the state-of-the-practice review discussed in Section 2.

To properly use this document, it is necessary to understand the assumptions used during the development of the methodology. First, this methodology is aimed at assessing a building's life-safety level of performance; the corollary recommendation is that all buildings be strengthened to this minimum level. Life-safety in this work has been broadly defined as damage that could kill an occupant, cause injury to the point of immobility, or block any of the dedicated means of egress from the building.

Also, this methodology is intended to catalog experience from past earthquakes and incorporate it with widely accepted analysis and design techniques in a concise format. The procedure is intended to guide but not restrict the evaluating engineer so that consistent and fairly complete thinking will be used in seismic evaluations. The methodology is tailored to the evaluation of individual buildings rather than groups of structures. But, the format is presented such that preliminary screening will identify which, if any, items need additional analysis. In this way, structures that present a small enough risk to life safety may not require detailed analysis. Finally, the methodology has been developed for use throughout the nation.

The methodology has been designed as a guide for use by practicing Structural Engineers. The final decision regarding adequacy, need for further study, or need for strengthening still rests with the evaluating engineer. For this reason, the evaluation procedures should be applied, or at least supervised, by a knowledgeable Structural Engineer.

4.1 Introduction to the Methodology

Figure 4.1 is a flow chart depicting the steps to be followed in using this methodology. This figure depicts all of the actions and decisions that will occur during the seismic evaluation. This figure should be used as a reference for the proper sequence of steps to be followed. The following paragraphs provide a description of the basic portions of the methodology depicted in Figure 4.1.

After being hired by the building owner or other party, the first step the engineer takes in the evaluation process is to obtain all possible documentation of the existing structure. This documentation could include drawings, specifications, calculations, geologic reports, or other information concerning the design, construction, maintenance, additions, or modifications of the structure. These documents can be important sources of information to be used throughout the evaluation process. Section 4.2.1 discusses the use of existing documentation.

If a set of construction documents is obtained, the engineer should review this information before performing a site visit to verify the accuracy of these documents, and to examine the existing condition of the building. The site visit also may be necessary to provide supplementary information not covered in the drawings but required for the seismic evaluation.

If no documentation is available, as is often the case of for old or "unengineered" buildings, the initial site visit will include the preparation of preliminary sketches of the structural systems. These drawings should be of detail sufficient to identify the vertical and lateral force resisting structural systems, and to perform the preliminary evaluation. A discussion of the techniques and procedures to be used in site visits is presented in Section 4.2.2.

Once the drawings have been reviewed and/or the initial site visit has been performed, the engineer should classify the building construction type and identify the model building(s) to be used in the evaluation. Section 4.3 discusses the classification and building identification procedures to be used. Provisions are included for dealing with structures that do not fall into any of the specific model building classifications.

The next step in the procedure is to examine the information provided in the model building evaluation section(s) appropriate for the building under consideration. There is a section provided for each of the fifteen model buildings (see Sections 5 through 10). Each of these model building sections contains the following items:

1. A general description of the structural features of the model building.

2. A description of the performance characteristics exhibited by structures of this type during past earthquakes of severe intensity. A definition of the Modified Mercalli Intensity used and the related peak ground accelerations is given in Table 4.1.

3. References that describe specific examples of the performance of structures of this type in past earthquakes.

4. An explanation of the probable load paths for vertical and lateral loads and an estimation of the typical weights of building components. In an effort to facilitate understanding of the lateral load transfer mechanism, these descriptions assume the seismic loads are transferred from the diaphragms to the vertical elements and into the foundations. The actual loads are generated from ground motion and transferred into the structural elements.

5. A collection of statements, related concerns, and suggested procedures to be addressed in the evaluation. These evaluation issues (collection of statements with related concerns and suggested procedures) are also summarized in checklist format in Appendix C.

After becoming acquainted with the appropriate model building sections, the engineer should determine the basic seismic risk zone for the structure by examining the seismicity maps provided in Figure 3.1 of Section 3. The basic seismicity zones are designated as "Low" (Effective Peak Acceleration, A_a (EPA) ≤ 0.10 g), "Moderate" (.10 g < EPA $\leq$ .20 g), and "High" (EPA > .20 g).

The set of statements and related concerns to be addressed in the evaluation depends on the basic seismicity of the building location. For regions of "Low" seismicity, a complete set of statements is presented that applies for each specific building type. These sets of statements were significantly expanded and improved during the NCEER Review Project of ATC-14. For regions of "High" seismicity, a more detailed and stringent set of statements and concerns must be addressed. The evaluation of buildings in regions of "Moderate" seismicity uses the same set of statements as that of "High" seismic zones. Since this report specifically addresses only regions of low seismicity, the set of statements for areas of high and moderate seismicity have not been included. The reader is referred to the original ATC-14 document for the evaluation of buildings in these seismic regions. Note that in the Central and Eastern United States, only the New Madrid region has a seismicity considered "moderate" for the purposes of this document. This report can therefore be used in the seismic evaluation of buildings in all other regions of the eastern and central United States.

It was felt by the ATC Project Engineering Panel that in regions of low seismicity, only the basic elements of a lateral force resisting system was necessary. For this reason, the shorter set of basic concerns was developed. The NCEER review group felt that more detailed and explicit sets of statements were required. These more detailed sets of statements are included in this report. For regions of moderate to high seismicity all of the same issues are appropriate and, therefore, the same statements are applicable. The difference comes during any required analytical evaluations when the lower force level is used.

The next step in the procedure is to address the set of statements and concerns that are appropriate for the building type and local seismicity of the structure being evaluated. Each statement is written such that a positive or "true" response implies that the building is adequate in that area. All of the statements have been provided with a short commentary (titled "Concern") to explain why the statement was written and to further assist the engineer in dealing with the specified issue. Again, this methodology is intended to flag items of concern that should be addressed by the engineer. Satisfying the issues presented in the "Concern" should therefore take precedence over blindly meeting checklist statement requirements.

If the building under evaluation adequately meets all the stated concerns (a "true" response is determined for all statements), no further evaluation of the structure is required, unless the structure has any special features not covered in the methodology that could present life-safety hazards. For statements that are "false", additional evaluation is required. This does not necessarily imply that a complete structural evaluation is necessary, or that the building is automatically deficient. In fact, the suggested procedure limits the evaluation to only the area of concern. It is offered as a suggested procedure since the responsibility for the evaluation rests with the Structural Engineer, who may elect to perform an alternate evaluation procedure. This is permissible as long as it addresses and leads to an opinion regarding the issue raised in the statement. Deficiencies are identified only after an appropriate detailed evaluation has been made.

The procedures suggested are based on the ATC-3 and ATC 6-2 type approaches to considering the capacity of the element under review and the demand place on that element. Section 4.4 presents the analysis procedures that are recommended in the building evaluation statements. This section includes a discussion of rapid evaluation techniques for preliminary screening, the suggested equivalent lateral force procedure, a dynamic analysis procedure, and a special analysis method for wood diaphragms. An allowable Capacity/Demand (C/D) ratio is listed for each statement that is based on the anticipated excess capacity available in the element and the level overall system ductility assumed in the demand criteria.

At any stage of the evaluation procedure where additional information is necessary, the engineer should return to the building site. The additional information can be acquired from more detailed site investigations and/or materials tests of critical structural elements. A guide to appropriate materials testing programs is provided in Section 4.2.3.

During the evaluation of the building, the engineer should also be concerned with detecting any possibly hazardous special features of the building that have not been included in the statements or concerns. This methodology is intended to cover the items of concern that are characteristic of the type of building under evaluation. Addressing unusual building features that could be hazardous should follow the spirit of this procedure.

Damage to nonstructural elements may also be of concern to the building owner. If this is the case, the nonstructural evaluation procedures presented in Section 11 can serve as a guide to the performance of nonstructural elements and the items to be concerned within the evaluation. This section is structured like the building evaluation sections/Sections, except that no concerns are included to explain the background of the checklist statements. As a result of the NCEER review project, this section has been expanded to include a complete list of cladding elements, a section on building contents, and a set of explanatory figures. Nonstructural elements that could pose a life-safety hazard, such as parapets or unbonded masonry veneer, have been included in the building evaluation procedures whenever possible. Evaluation of the requirements for nonstructural elements can be performed using the procedures given in Section 4.4.5. If special circumstances result in other possible threats to life safety from nonstructural elements, they should be noted in the engineer's report to the owner.

The final step in the evaluation procedure is to report the results to the owner or client. This step will usually consist of a written report that details the findings of the investigation, including a listing of all (if any) possible life-safety hazards that may exist. If nonstructural considerations are also investigated, concerns for damageability to these items should also be included. The report may also include suggested methods to retrofit or strengthen the building to mitigate these hazards.

4.2 Data Collection Procedures

Collection of the necessary information is a critical portion of the seismic evaluation of any existing building. The information gathered in this step becomes the basis for both the identification of the appropriate building type and the evaluation of the statements presented in the procedure. But, data collection does not only occur in the early stages of the evaluation procedure. Rather, it may occur at any stage of the evaluation, whenever additional information is required.

The sources of data to be used in the seismic evaluation of an existing building include the following three items:

1. Contract documents,such as construction drawings, specifications, soils reports, and calculations

2. Field surveys of the structure's existing condition

3. Destructive and nondestructive tests of structural materials

4.2.1 Existing Drawings and Reports

Perhaps the best source of information for performing a seismic evaluation is a complete set of construction drawings. The drawings provide information on the connections and other details that are typically concealed by architectural finishes or structural materials. The existence of complete structural drawings greatly simplifies and reduces the amount of field survey work required. Beside basic information about the structural framing, the drawings may also include information on the design code, soil bearing values, and assumed loadings. Another important construction document that may be useful in the seismic evaluation are the project specifications. The specifications can provide useful information on the material properties, inspection procedures, and other quality control measures used during the original construction. Information on the architectural, mechanical and electrical systems that may be relevant to the seismic evaluation can often be found in the specifications. The calculations made in the design of the original structure can also be used as a source of information. These calculations may be used to determine the base shear and the basic assumptions used in the original design. The engineer(s) performing the evaluation may choose not to examine the original calculations until a separate analysis has been performed. If areas of concern are identified in the evaluation, the original calculations may be consulted to determine how these items were addressed in the original design. By performing a separate analysis before consulting the original design calculations, the evaluator(s) are assured that their results will not be biased by the design engineer's assumptions. In any case, the results of the original design calculations should be compared with the construction documents and the actual building. If these items are not in agreement, the engineer performing the evaluation should try to determine how the changes will alter the seismic response of the building.

The final document that may be of use in the seismic evaluation is the geotechnical report that may have been performed for the original design. This report may include information concerned with the local geology, water table, and seismicity of the site. Bearing capacity values for both gravity and lateral loads may also be provided in this document.

The design and construction documents can provide a tremendous source of information. The engineer(s) performing the evaluation should therefore make every effort to obtain as many of these documents may not be available for old or "unengineered" buildings. The following list contains possible sources of these documents:

- The building owner and/or previous owners

- The original architect and/or engineer

- The local jurisdiction or city building department

- The building mechanical room or building engineer's office

- Local, state, or national historical societies for buildings deemed to have historical significance

Note that when dealing with large organizations, such as government agencies, it may be necessary for the engineer to perform a personal search of the organization archives to locate the appropriate building documents.

4.2.2 Field Investigation of Existing Conditions

Another vital step to be followed in information collection is to visit the site to investigate field conditions. The number and degree of the field investigations depends on the completeness of information provided by the contract documents.

If a complete set of construction drawings and other documents is provided, it may be necessary to visit the site only in the initial stage of the evaluation. This initial survey should always be done to verify the accuracy of the documents because field changes or building additions may significantly alter building response. This verification process may include the removal of architectural finishes in certain areas to expose key connections or other details. Note that before removing any finishes, the engineer should obtain specific permission from the building owner and arrange for the restoration of the disrupted areas. Another important item to be evaluated in the initial site visit is the condition of the structural materials. Without proper maintenance, material deterioration can occur, which may result in serious strength deficiencies. Types of material deterioration that should be investigated include dry rot or infestation of wood, corrosion of steel, cracking or spalling of concrete, and cracking of masonry.

In addition to verification and examination of present material conditions, the initial site visit and/or document examination should uncover the following general information:

- Name and address of the structure

- Age of original structure and any additions or alterations, and the applicable building code

- The number of stories and basements, and the story heights

- Any important exterior wall features such as parapets, appendages, ornamentation, setbacks, and/or roof overhangs

- Proximity and condition of adjacent structures, and their potential for damage to the building being evaluated

- Building use and occupancy, and any changes in occupancy type that may have occurred

If no drawings are available, or if the documents are incomplete, more extensive field investigations will be required and schematic as-built drawings should be prepared. These drawings should include a separate plan that shows the location and size of shear walls and/or frames for all levels with appreciable differences. These plans should also note the size and location of expansion or separation joints, the existence of any party walls, and the typical floor framing scheme. Other drawings should consist of interior and exterior wall elevations that show all wall openings, materials, major cracking and repaired damage. All changes in thickness, discontinuities and/or areas of material deterioration should also be noted. Sketches of typical and special details and connections, which are part of the gravity or lateral load resisting systems, should also be made.

The purpose of preparing these drawings is to provide the information necessary to perform the seismic evaluation. Therefore, it behooves the engineer to be thoroughly familiar with the information in the evaluation procedure for the appropriate building type(s). This knowledge may save the engineer from performing site visits that could have been avoided.

In addition to verifying and/or preparing drawings to be used in the evaluation and assessing the present condition of building materials, the site investigation should include checking the following features:

- Additions to the building, such as mezzanine levels or attached structures
- Alterations to structural elements, such as new openings in shear walls, or openings filled with unreinforced masonry
- Anchorage of concrete or masonry walls to roof and floor diaphragms
- Masonry infills in steel or concrete frames, including the provision of isolation from lateral displacements
- Parapets, including the height, thickness, materials, anchorage and location
- Unreinforced masonry walls, including coursing, bonding condition of mortar and material of each wythe
- Masonry veneers on exterior walls, including anchorage, presence of shelf angles and provision for relative movement
- Longitudinal splitting (checks) in wood framing elements
- Gable ends of unreinforced masonry walls, including size
- Any special features that could create a hazard or affect the seismic response

The gathering of information on nonstructural items that can create a seismic hazard should also be included as part of the site investigation. Some of the items about which information should be gathered include:

- Partitions, both typical and corridor (identify material, extent, and connections to structural elements)
- Ceilings, both typical and corridor (identify material and connections)
- Large mechanical and electrical equipment (check for anchorage to the structure)
- Light fixtures (check anchorage to ceiling or structure)
- Gas lines and other piping (check lateral bracing elements)

In performing a site investigation, the engineer should examine all exterior wall elevations and the major corridors. In addition, it may be helpful to talk with the owner, building engineer, and maintenance personnel of the building about its condition and features. The following areas of the building may not be covered by architectural finishes and could therefore provide useful information on the structural features:

- The regions between suspended ceilings and the floors above, where there may be openings in structural elements for mechanical ductwork (these areas can be reached by removing ceiling tiles or using access hatches)
- Mechanical rooms, shafts, or pipe chases
- Elevator shafts
- Crawl spaces below the first floor level
- Roof
- Areas under stairways

The following equipment may be useful in the site visits:

- Camera, with wide angle and telephoto lenses
- Tape measure
- Scaffold or binoculars to examine tall exterior walls
- Ladder
- Geologist's hammer
- Crack width comparator
- Notebook or tape recorder

A field data sheet has been developed to assist in recording general information to be gathered during the initial site visit and data collection. See Section 12 and Appendix C for more information on this sheet and its application.

4.2.3 Testing Methods

In addition to examination of construction documents and performing site investigations, the engineer may gain useful information by performing tests on the structural materials. The extent to which testing should be utilized depends on the type of material, local seismicity, uniformity of materials, amount of observed deterioration, and criticality of the structural elements. Testing of building materials will often be most useful after a preliminary analysis determines the possibly critical areas. The appropriate tests will be dealt with separately by material.

For wood structures, decay detection can be accomplished by visual examination, sounding with a hammer, drilling, coring, ultrasonic pulse velocities, or electrical resistance. Moisture content can be estimated with instruments that measure electrical resistance. If visual inspection of wood members does not uncover the appropriate grading, the engineer may wish to have key elements regraded to better estimate the allowable stresses. Glue joints in laminated members may be tested by removing a cylindrical plug. See Freas (1982) for further discussion of testing and inspection of wood structures.

Testing of structural steel in the evaluation of existing structures is typically only done in special cases. In these cases, steel coupons may be removed from noncritical areas of structural elements to obtain estimates of the yield and tensile capacities and the ductility of the material. Typically, a fairly accurate estimate of the yield strength can be obtained from the type of steel and the age of the building. This should suffice in most cases.

A large number of tests have been developed to estimate the properties of existing concrete structures. Compressive strength is generally estimated by testing a series of concrete cores. A number of nondestructive tests, including those with rebound hammers, probe-penetration devices, ultrasonic pulse velocity, and pullout tests, have also been developed to test in situ concrete. These tests should not be thought of as substitutes for standard core tests due to the variability of results. However, these tests can be combined with core tests to reduce the cost of testing because they can fairly accurately determine the uniformity of concrete. Other tests have been developed to examine different properties of reinforced concrete. Pachometers are widely used to locate and estimate the depth of reinforcing steel. Other concerns such as cracking, existence of voids, condition of reinforcing steel, and moisture can be estimated by some recently developed and sometimes highly sophisticated techniques. It is generally advisable for the tests to be performed and interpreted by skilled technicians. Two references (Malhotra, 1976; Bungey, 1982) provide excellent descriptions of the available tests in must grater detail than can be presented here.

Recent experimental investigations (Noland et al., 1982) have been performed on masonry structures using many of the nondestructive tests developed for concrete testing. This work also concludes that the nondestructive tests should be combined with destructive tests. The results of these experiments indicate that the nondestructive test methods (rebound hammer, penetrometer, pulse velocity, etc.) are more applicable to recently constructed masonry.

For older masonry, such as that found in unreinforced bearing wall buildings, the use of in-place shear tests has been prescribed for strength estimations by earlier methodologies (ABK, 1984). The ABK methodology presents the suggested number and location of such tests. In this test, a single brick is laterally displaced to provide an estimate of joint shear strength. This procedure is favored over masonry core samples because of the large variability exhibited by core tests (Noland et al., 1982). In addition to masonry wall testing, the anchorage of unreinforced bearing walls to the diaphragms should be tested. The ABK methodology presents a suggested anchorage testing procedure that is based on an existing testing program now in effect in Los Angeles, California (SEAOSC, 1981).

4.3 Building Identification System

The building identification system employed in this methodology is based on the material or type of construction employed in the principal gravity and lateral force resisting elements. The number of possible combinations of materials and structural systems that can be employed in building construction in the United States is extremely large. But not all of the possible combinations are economical enough to be widely used. This fact makes it possible to identify the typical building types that are or have been prevalent in different areas of the United States. A total of 98 different building types were identified. These 98 building types may not form a completely exhaustive list, but they do include the vast majority of existing buildings. The development of a separate methodology for the seismic evaluation of each of these 98 building types would be extremely lengthy and, therefore, impractical for use by the practicing engineer. Fortunately, regularly configured buildings that employ like construction materials and/or structural systems have exhibited similar performance characteristics in past earthquakes. It was, therefore, possible to identify a subset of 15 model building types from the total inventory of building types. These model building types were developed to incorporate as many of the structural and performance characteristics of the total inventory as possible. By reducing the number of building types to only those necessary to exhibit the basic structural and performance characteristics, it was possible to develop a seismic evaluation methodology that treats each model building type uniquely. To simplify the evaluation procedure, these model buildings were categorized by their structural materials. The six material categories are: (1) wood, (2) steel, (3) cast-in-place concrete, (4) precast concrete, (5) reinforced masonry, and (6) unreinforced masonry. A list of the model building types is presented in Table 4.2. Table 4.3 forms the basis for identifying which model building evaluation procedure(s) should be employed in any seismic evaluation. This table provides a matrix of possible building combinations. At the top of the table is a list of six typical types of structural diaphragms. Along the left side of the table are possible wall and/or framing types. Each element of the matrix, therefore, represents a possible combination of diaphragm and wall and/or framing elements.

The procedure to be followed in determining the model building type(s) includes the following steps:

1. Obtain all available drawings and other building documents. Examine these to determine the basic vertical and lateral force resisting systems.

2. If no drawings of the structure are available, perform an initial site visit to become familiar with the building configuration and construction.

3. Identify the gravity load resisting system and determine the load path.

4. Identify the elements of the lateral load resisting system and determine the load path. Identify both the horizontal and vertical elements that resist lateral displacements. Note if different systems are used in orthogonal directions or in different levels.

5. Use the designation of horizontal and vertical lateral force resisting elements established in Step 4 above with Table 4.3 to determine the most appropriate model building type(s) to be employed in the seismic evaluation.

Three possibilities exist for the number of model building designations found in any matrix element that represents a single building type in Table 4.3:

1. A single model building type designation

2. Multiple model building type designations

3. No model building type designation

Matrix elements in Table 4.3 with single building designations should result in the most straightforward evaluation because only one model building evaluation procedure needs to be considered. In this case, one of the model buildings should adequately portray the seismic performance characteristics of the structure being evaluated.

It is expected that typically a single building will apply for both major axis directions of a structure. But in some cases, the lateral force resisting system for the two principal directions may be different. In such cases, a multiple model building type designation is required. One common example of this situation is a steel framed building with moment resisting frames in the longitudinal direction and braced frames in the transverse direction. For this case, a separate evaluation, using the appropriate model building types, should be performed for each of the principal directions. If a detailed analysis of the system is required, both principal directions and their interaction need to be considered.

Multiple model building type designations are also required for buildings having diaphragm and wall and/or framing combinations that have not been identified as one of the 15 model building types and for structures with structural; systems that change with the height of the structure. In the case of buildings having unusual diaphragm, wall and/or framing combinations, two model building designations usually are necessary, one for the wall or framing type and one for the diaphragm construction. In structures having structural systems that change with height, such as, for example, a structure with a wood frame residential building above a concrete parking garage structure, the compatibility of elements of the two systems will require special attention. In performing an evaluation of buildings of either type, the engineer should combine statements from each of the applicable model building types into a set of statements that are appropriate for the structure under consideration. Statements in either of the model building types that are not relevant to the form of construction being evaluated need not be considered. It should be emphasized that when performing an evaluation of a structure that requires the combination of statements from two model building types, the engineer must be watchful for any special concerns that may result from this type of building construction.

In those cases where Table 4.3 does not include any appropriate model building designation, the construction type has been designated by its absence not to be popular in the United States due to economic, structural, or other reasons. If such a building is encountered, the engineer will need to exercise careful judgment in employing this methodology. Many of the principles that are basic to any seismic evaluation will still be applicable, but it will be necessary to combine and consider various portions of the methodology. An example building type in this category is the "lean-to" structure. Such structures, which are added adjacent to a previously existing building, rely on the lateral capacity of the existing structure, and careful examination of the connection between the original structure and the addition must be performed. In general, careful professional judgment will be required, and in some cases it may be necessary that the evaluation be conducted by an engineer with substantial experience in seismic analysis and design.

4.4 Analysis Procedures

The building evaluation procedures presented in Section 5 through 10 consist of sets of "Statements" that identify items of concern (these statements are also summarized in abbreviated checklist format in Appendix C). When the building does not meet the requirements of any "Statement" or the related "Concern", a more detailed analysis of this building feature may be recommended. This section presents a discussion of the analysis procedures recommended in the "Procedure" portions of the evaluation methodology.

Many of the "Procedures" include recommendations for calculating capacities and demands on certain structural or nonstructural elements. Capacities can be calculated using the recommendations of Section 4.4.1, which are generally based on the provisions of the applicable building code requirements for the structural material being considered. Demand calculations can be performed with the aid of Sections 4.4.2 through 4.4.5. Section 4.4.2 discusses the procedures to be used in performing a rapid evaluation. Sections 4.4.3 and 4.4.4 contain equivalent lateral force and dynamic analysis procedures, respectively. Section 4.4.5 addresses the lateral force evaluation of elements of structures and nonstructural components. Section 4.4.6 presents a suggested special analysis procedure for wood diaphragms that is based on recent research work performed by the ABK Joint Venture (ABK, 1984).

The procedures recommended for calculating the capacities and demands were intended to provide appropriate, widely recognized analysis methods, that would not require specialized expertise. Other analysis procedures that employ new, advanced, or specialized techniques may also be used when applicable. One such example would be the use of the recommendations developed by the Seismic Committee of the New Zealand Prestressed Concrete Institute (1976) for the seismic analysis of prestressed concrete frames. All special analysis procedures that are employed must still comply with the basic intent of the specific "Statement", "Concern" and "Procedure" presented in the building evaluation methodology.

The Capacity/Demand (C/D) ratios can be calculated from the values computed using the recommendations of Sections 4.4.1 through 4.4.6, or other appropriate procedures. This value should then be compared with the "Recommended C/D Ratio" given for the statement being investigated. The C/D ratios recommended are typically either given a numerical value or a fraction of the working stress structural response modification (R_w) factor of the building being evaluated (0.2 R_w or 0.4 R_w). The engineer may also justify use of C/D ratios other than the recommended value. But in no case should the C/D ratio be less than 1.0, except as specified for wood diaphragms in Section 4.4.6. This may occur for low values of R_w.

4.4.1 Calculation of Member Capacities

In many of the recommended procedures, the calculation of member capacities is required. These capacities should generally be calculated using the appropriate building code provisions for the structural material being evaluated. In other cases, materials testing performed in accordance with the recommendations of Section 4.2.3 may be necessary.

The basic resource documents to be used in calculating the member capacities for various structural materials are listed below. Appendix F contains a complete list of the mailing addresses needed to obtain these documents.

Material	Resource Document
General:	Uniform Building Code and Uniform Building Code Standards (ICBO, 1991), Standard Building Code (SBC, 1991), and the BOCA National Building Code (BOCA, 1991)
Wood:	Section 25 of the Uniform Building Code (ICBO, 1991), including the 1/3 stress increase on the allowable stresses. Also, see National Design Specification (NFPA, 1986) and the Timber Construction Manual (AITC, 1986)
Steel:	American Institute of Steel Construction Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings (AISC, 1989), including the 1/3 stress increase on the allowable stresses. Also, see LRFD specifications (AISC, 1986), and Cold formed Steel Design Manual (AISI,1986), Structural Welding Code (AWS, 1991), Standard Specifications for Steel Joists and Girders (SJI, 1988), and Diaphragm Design Manual (SDI, 1986)
Concrete:	Section 26 of the Uniform Building Code (ICBO, 1991). The combination of Dead (DL), Live (LL), and Seismic (E) loads used to calculate demand values in the C/D ratios for concrete is as follows: $U = 1.4\ (DL + LL + E)$ or $U = 0.9DL \pm 1.4E$ where U = Required strength to resist factored loads. Also, see Manual of Concrete Practice (ACI, 1988g), PCI Design Handbook (PCI, 1985), Post-Tensioning Manual (PTI, 1985), PCI Manual on Design of Connections (PCI, 1973)
Masonry:	Section 24 of the Uniform Building Code (ICBO, 1991), including the 1/3 increase on the allowable stresses. Also, see Building Code Requirements for Masonry Structures (ACI, 1988b), Building Code Requirements for Engineered Brick Masonry (BIA, 1986) and a Design Manual for Load Bearing Walls (NCMA, 1985).

Other methods for calculating member capacities that can be justified by the engineer may also be employed.

4.4.2 Rapid Analysis Procedure

The seismic performance of existing buildings depends to a significant extent on the amount of strength and/or drift control provided by the elements in the lateral force resisting system. In regions of high or moderate seismicity, the seismic evaluation of most types of existing buildings should include at least a rapid check of the lateral strength and/or the interstory drift. A similar rapid check may also be included in the seismic evaluation of buildings in regions of low seismicity, if the engineer believes that special conditions warrant such an investigation.

The rapid evaluation procedure presented in this section is intended to be combined with the rapid evaluation procedures recommended for the various building materials in Sections 5 through 10. For shear-wall structures that rely on structural walls of wood, concrete, reinforced masonry, or unreinforced masonry infills, the rapid evaluation procedure will include checking the average shear stress. Steel and concrete moment-frame structures will be checked for interstory drift. Rapid evaluation of concrete frame buildings will also include a check of the shear stress in the columns, and for steel braced frame buildings, it will include a check of the axial stress in the diagonal braces.

The base shear to be used in the rapid evaluation procedure should be determined from the following equation:

$$V = \frac{2.5A_a}{R_w} W \tag{4.1}$$

where: V = Base shear of the building

A_a = Effective Peak Acceleration given in Figure 3.1

R_w = Numerical coefficient given in Table 4.4

W = Total seismic dead load defined in Section 4.4.3

In reviewing an existing structure, it may be necessary to check the average shear stress or drift for upper stories in addition to the first story. In this case, the story shear for any story, can be approximated by the following formula:

$$V_j = \frac{n + j}{n + 1} \frac{W_j}{W} V \tag{4.2}$$

where: V_j = Maximum story shear at story level j

n = Total number of stories above ground level

j = Story level under consideration

W_j = Total seismic dead load of all stories above level j

W = Total seismic dead load

V = Base shear determined using Equation 4.1

It should be noted that these equations are applicable only for structures less than six stories in height. For taller structures, a rapid check of the story shear strength is not sufficient because such structures may be governed by overturning. Taller structures should be evaluated using either the equivalent lateral force or dynamic analysis procedure.

Once the V and V_j terms have been determined, the engineer should compute the average level of stress or interstory drift as suggested in the evaluation procedure under consideration. The calculation of average stress levels should be limited to the lateral force resisting elements with consideration given to the overall system behavior. This average stress or drift level should then be compared with the minimum allowable value given in the evaluation procedure. If the allowable stress is less than this average value, a more detailed evaluation of the walls and/or frames should be performed. Recommended procedures to be used in this more detailed evaluation are included in the specific building discussions

Satisfying the average drift or stress check discussed in this section does not automatically relieve the building from any further evaluation. The "Statements" and "Concerns" given in the remainder of the specific building discussions must still be addressed in all building evaluations.

The coefficient 2.5 in Equation 4.1 is 20 percent larger than the maximum C value in the equivalent lateral force procedure of Section 4.4.3. This conservatism has been added to account for such items as: (1) possible lower construction standards of older buildings; (2) possible deterioration of structural elements; (3) torsional effects; and (4) mass or geometric irregularities. If the building does not satisfy the requirements of the rapid stress or drift evaluation, these and other items should be accounted for in the more rigorous subsequent analysis. The simplified formulas given in this section should not be used for structures over six stories because taller structures will have significantly different lateral force levels and distributions. As previously indicated, taller structures should be evaluated using either the equivalent lateral force or the dynamic analysis procedures.

4.4.3 Equivalent Lateral Force Procedure

This section presents the requirements of the equivalent lateral force procedure that is recommended for the detailed analysis of existing buildings. This procedure should be used to develop the seismic demands on the elements of the lateral force resisting system identified by the building evaluation procedure(s) as requiring further investigation.

The general requirements of the analysis procedure, including the determination of force level, horizontal distribution of lateral forces, accidental torsion, interstory drift, and overturning, are included in this section. The recommended procedure is based on the material presented in Section 3 as well as in the Structural Engineers Association of California Tentative Lateral Force Requirements (SEAOC, 1985).

The lateral force level inherent in these provisions has been reduced from the level specified int he Tentative Lateral Force Requirements to match the response spectra given in Section 3. The force level given, then, is based on a median response spectra for an expected ground motion with a 10 percent chance for exceedance in fifty years. The difference is evident in the formula for the coefficient C in Equation 4.4, and the related maximum values.

It was the consensus opinion of the Project Engineering Panel that this minimum lateral force level represented the minimum strength needed by a building to provide life-safety protection. In recent years, however, the appropriateness of this minimum strength level has been questioned and lower levels have been suggested based on the expected ground motion with a larger probability of exceedance and/or a shorter recurrence interval. If the governing jurisdiction agrees to such a lower level, the related coefficients in this equivalent lateral force procedure may be calculated according to the recommendations provided in Section 3.

Only the basic provisions of the analysis procedure are included here; the individual building evaluation procedures presented in Sections 5 through 10 identify the specific items that require consideration. This section should therefore be used in conjunction with the building evaluation procedures for specific building types to address the items of concern and to calculate the appropriate C/D ratios.

A. Criteria Selection

1. Basis of Design. The procedures and limitations for the seismic evaluation of buildings should be determined considering zoning, site characteristics, occupancy, configuration, structural system, and height in accordance with the requirements of this section.

2. Seismic Zones. Each site should be assigned to a seismic risk zone. The maps of Figures 3.1 and 3.2 provide the Effective Peak Acceleration, A_a, and the Effective Peak-Velocity-Related Acceleration, A_v.

3. Site Geology and Soil Characteristics. Soil profile type and site coefficient, S, should be established in accordance with Table 4.5.

4. Occupancy Categories. For purposes of seismic evaluation, each structure should be placed in one of the occupancy categories listed in Table 4.6. Table 4.7 lists importance factors for each category.

5. Selection of Lateral Force Procedure. The static force procedures of this section prescribe the minimum seismic forces. Any structure may be evaluated, and certain irregular structures identified in the appropriate model building evaluation procedure should be evaluated using the dynamic procedures of Section 4.4.4.

6. Tall Structures. All buildings 240 feet in height and taller should be evaluated using the dynamic lateral force procedures of Section 4.4.4.

7. Alternate Procedures. Alternate lateral force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these recommendations.

B. Minimum Design Lateral Forces and Related Effects

1. General.

a. Seismic force should be assumed to come from any horizontal direction.

b. The seismic forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure, except as required by Paragraph B.8.c of this section.

c. Seismic dead load, W, is the total dead load and applicable portions of other loads listed below.

   (1) In storage and warehouse occupancies, a minimum of 25 percent of the floor live load is applicable.

   (2) If applicable, a partition load of not less than 10 psf is to be included. A partition load of 20 psf may be appropriate for plaster walls.

   (3) Where the snow load is greater than 30 psf, the snow load is to be included. Where considerations of siting, configuration and load duration warrant, the snow load may be reduced up to 75 percent.

   (4) Total weight of permanent equipment is to be included.

2. <u>Equivalent Lateral Force Procedure.</u>

   a. <u>Design Base Shear.</u> The total base shear in a given direction should be determined from the following formula:

$$V = \frac{(A_v)\, ICW}{R_w} \qquad (4.3)$$

where: A_v = Effective Peak-Velocity-Related Acceleration given in Figure 3.2

I = Importance factor given in Table 4.7

R_w = Numerical coefficient given in Table 4.4

W = The total seismic dead load defined in this section

C = Numerical coefficient determined from the following formula:

$$C = \frac{0.80S}{T^{2/3}} \qquad (4.4)$$

where: S = Site coefficient for soil characteristics given in Table 4.5

T = Fundamental period of vibration, in seconds, of the structure for the direction under consideration determined in accordance with the following paragraphs.

The ratio, C/R_w, should be not less than 0.05. The value of CA_v need not exceed $2.12A_a$, where A_a is the Effective Peak Acceleration given in Figure 3.1. A value of $C = 2.12$ may be used for any structure without regard to soil type or structure period.

b. Structure Period. The value of T should be determined from one of the following methods:

(1) Method A:

For all buildings, the value of T may be approximated from the following formula:

$$T = C_t(h_n)^{3/4} \tag{4.5}$$

where: C_t = 0.035 for steel frames

= 0.030 for reinforced-concrete frames

= 0.020 for all other buildings

h_n = Height, in feet, above the base to Level n

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$.

The value of A_c shall be determined from the following formula:

$$A_c = \Sigma A_e[0.2 + (D_e/h_n)^2] \tag{4.6}$$

The value of D_e/h_n used in formula shall not exceed 0.9.

where: A_e = the minimum cross-sectional shear area in any horizontal plane in the first story, in square feet, of a shear wall.

D_e = the length, in feet, of a shear wall in the first story in the direction parallel to the applied forces.

(2) Method B: The fundamental period T may be estimated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. This requirements may be satisfied by using the following formula:

$$T = 2\pi \left(\sum_{i=1}^{n} w_i\delta_i^2 \Big/ g\sum_{i=1}^{n} f_i\delta_i\right)^{1/2} \tag{4.7}$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Equations 4.8, 4.9, and 4.10, or any other rational distribution. The elastic deflections, δ_i, should calculated using the applied lateral forces, f_i. The value of C (Equation 4.4) should be not less than 80 percent of the value obtained by using T from Method A above.

3. Combinations of Structural Systems. Where combinations of structural systems are incorporated into the same structure, the following requirements should be satisfied:

   a. Vertical Combinations.

      (1) The value of R_w used in the design of any story should be less than or equal to the value of R_w used in the given direction for the story above.

      Exception: This requirement need not apply to stories that support weight less than 10 percent of the total dead weight of the structure.

      (2) Structures not having the same structural system throughout their height should be evaluated using the procedures of Section 4.4.4.

      Exceptions:

      (a) Structures five stories and under may be evaluated using the procedures of this section.

      (b) Structures conforming to paragraph (3) below.

      (3) A two-stage analysis may be used where a structure contains a rigid base supporting a flexible upper portion such as concrete parking garages that support wood frame dwellings, and both portions considered separately can be classified as regular structures. The rigid base should have a calculated natural period in each direction of not more than 0.06 seconds. The periods should be evaluated using Equation 4.7 to its equivalent, considering the total mass of the flexible upper portion concentrated at the top of the rigid base.

      (a) The flexible upper portion should be evaluated as a separate structure, supported laterally by the rigid base, using the appropriate value of R_w.

      (b) The rigid base should be evaluated as a separate structure using the appropriate value of R_w. The reactions of the flexible upper portion should be increased by the ratio of the R_w values of the two portions. These factored reactions should be applied at the top of the rigid base in addition to the forces determined for the base itself.

   b. Combinations Along Different Axes. If a building has a bearing wall system in only one direction, the value of R_w used for the other direction should not be greater than that used for the bearing wall system.

4. Vertical Distribution of Forces. The total seismic force should be distributed over the height of the structure in conformance with Equations 4.8, 4.9 and 4.10.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (4.8)$$

The concentrated force at the top, F_t, should be determined from the formula:

$$F_t = 0.07TV \qquad (4.9)$$

F_t need not exceed 0.25V and may be considered as zero where T is 0.7 seconds or less. The remaining portion of the base shear should be distributed over the height of the structure, including level n, according to the following equation:

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (4.10)$$

At each level designated as x, the force F_x should be applied over the area of the building in accordance with the mass distribution at that level. Stresses in each structural element should be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

5. Horizontal Distribution of Shear. The story shear, V_x, should be distributed to the various elements of the vertical lateral force resisting system in proportion to their rigidities, considering the rigidity of the diaphragm.

6. Horizontal Torsional Moments. The increased shears resulting from horizontal torsion where diaphragms have the capability to transmit that torsion should be evaluated. The torsional moment at a given story is the moment resulting from eccentricities between applied lateral forces at levels above that story and the vertical resisting elements in that story plus an accidental torsional moment. The accidental torsional moment should be determined assuming displacements of the centers of mass each way from their calculated locations. The minimum assumed displacement of the center of mass at each level can be estimated to equal 5 percent of the dimension at that level measured perpendicular to the direction of the applied force. For each element, the more severe loading should be considered.

7. Overturning. Every structure should be capable of resisting the overturning effects caused by earthquake forces specified in paragraphs 2 and 4, above. At any level, the overturning moments to be resisted should be estimated using those seismic forces (F_t and F_x) that act on levels above the lever under consideration. At any level, the incremental changes of the overturning moment should be distributed to the various resisting elements in the manner prescribed in paragraph 5, above. See the following paragraph for combining gravity and seismic forces.

8. Combination of Forces. The individual components should be capable of resisting the prescribed seismic loads acting on them. The components should also comply with the specific requirements given in the appropriate code provisions for that material. In addition, such framing systems and components should comply with the following requirements:

   a. Combined Vertical and Horizontal Forces. All building components should be able to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live, and snow loads.

   b. Uplift Effects. Consideration should be given to uplift effects caused by seismic loads. Dead loads should be multiplied by 0.85 when used to resist uplift.

   c. Orthogonal Effects. Consideration should be given to the effects of earthquake forces acting in a direction other than the principal axes. This requirement may be satisfied by evaluating such elements for 100 percent of the prescribed seismic forces in one direction plus 30 percent of the prescribed forces in the perpendicular direction.

9. Story Drift Limitation. Story drift is the displacement of one level relative to the level above or below due to the lateral forces. Estimated drift should include translational and torsional deflections. Estimated story drift should not exceed 0.005 times the story height unless it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety. The lateral forces used to determine the calculated drift may be derived from a value of C obtained from Equations 4.7 and 4.4 neglecting the 80 percent limitation of Paragraph B.2.b.(2) of this section.

10. P-Delta Effects. The resulting member forces and moments and the story drifts induced by P-delta effects should be considered in the evaluation of overall structural frame stability. P-delta need not be considered if the drift limitations in Paragraph 9, above, are not exceeded.

11. Foundations. The foundation should be capable of transmitting the base shear and the overturning forces defined in this section from the structure into the supporting soil, but the short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

    a. Soil Capacities. The capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier, or caisson and the soil should be sufficient to support the structure with all prescribed loads, other than earthquake forces, taking due account of the settlement that the structure is capable of withstanding. For the load combination, including earthquake, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short time of loading and the dynamic properties of the soil. Allowable soil stresses may be increased by more than 33 percent of the allowables if substantiated by geotechnical data. For piles, this refers to the pile capacity as determined by pile-soil friction.

b. Structural Materials. The strength of foundation components subjected to seismic forces along or in combination with other prescribed loads and their detailing requirements should be determined from the provisions of ACI 318 (ACI, 1989).

4.4.4 Dynamic Lateral Force Procedure

This section presents the suggested procedure for using dynamic analysis in the detailed evaluation of existing buildings. In most cases, the "Procedures" in the evaluation methodologies for each building type as described in Sections 5 through 10 suggest that the equivalent lateral force procedure be used. The use of the dynamic analysis procedure is only suggested for tall structures, buildings with vertical irregularities caused by significant mass or geometric irregularities, or other cases where the distribution of the lateral forces departs from that assumed in the equivalent lateral force procedure.

The suggested procedure uses elastic response spectra that can be developed on the basis of the information presented in Section 3. Site specific response spectra developed for the building under consideration may also be employed. Time history analysis has not been included in the procedure. The discussion presented here has been developed from the SEAOC Tentative Lateral Force Requirements (SEAOC, 1985). Structures that are evaluated using dynamic analysis procedures should also meet all other applicable requirements of Section 4.4.3.

A. General. Dynamic analysis procedures, where suggested, should conform to the criteria established in this section. The analysis should be based on an appropriate ground motion representation as specified in this section and should be performed using accepted principles of dynamics. Structures that are evaluated in accordance with this section should comply with all other applicable requirements of these recommendations.

B. Ground Motion. The ground motion representation may be one of the following:

   1. The response spectra from the construction procedure given in Section 3.

   2. Elastic response spectra developed for the specific site. The ground motion represented by the spectra should be based on the geologic, tectonic, seismic recurrence information and foundation material properties associated with the specific site. The spectra should be representative of motions that can be generated by all known faults affecting the site.

C. Mathematical Method. A mathematical model of the physical structure should represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate to calculate the significant features of its dynamic response. A three-dimensional model should be used when the dynamic analysis involves a structure with a highly irregular plan configuration and rigid or semi-rigid diaphragms.

D. Analysis Procedure.

1. Response Spectrum Analysis. An elastic dynamic analysis of a structure should use the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve that corresponds to the modal periods. Maximum modal contributions are combined in a statistical manner using recognized combination methods (e.g., SRSS or CQC) to obtain an approximate total structural response.

2. Scaling of Results.

   (a) When the base shear for a given direction is less than that required by the equivalent lateral force procedure, the base shear should be increased to the value prescribed in that section. All corresponding response parameters, including deflections, member forces, and moments should be increased proportionally.

   (b) When the base shear for a given direction is greater than that required by the equivalent lateral force procedure, the base shear may be decreased to the value prescribed in that section. All corresponding response parameters, including deflections, member forces, and moments may be decreased proportionately.

3. Directional Effects. Directional effects for horizontal ground motion should conform to the requirements of Section 4.4.3, Paragraphs B.1.a and B.1.b.

4. Torsion. The analysis should account for torsional effects, including accidental torsional effects as prescribed in Section 4.4.3, Paragraph B.6. Where three-dimensional models are used for analysis, effects of accidental torsion should be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 4.4.3, Paragraph B.6.

4.4.5 Lateral Forces on Elements of Structures and Nonstructural Components Supported by Structures

This section presents the requirements for elements of structures and nonstructural elements in buildings. These provisions should be used when the evaluation methodologies for each building type (Sections 5 through 10) or the nonstructural elements (Section 11) suggest that lateral force calculations of specific structural elements, such as diaphragms, or nonstructural items, such as parapets, chimneys, appendages, exterior cladding, equipment, are necessary. The force level defined in this section should be compared with the accelerations estimated from the equivalent lateral force procedure, with the maximum value being used in the analysis.

A. General. Parts and portions of structures and permanent nonstructural components, and equipment supported by a structure, and their attachments, as identified in the building evaluation procedures, and/or Section 11, should be evaluated to verify that they are capable of resisting seismic forces prescribed in this section. All attachments or appendages, including anchorages and required bracing, should be evaluated for seismic forces. Nonrigid equipment, the failure of which could cause a life-safety hazard, should also be evaluated (see Section 11).

Each element or component evaluated should be capable of resisting a total lateral seismic force, F_p, given by the following formula:

$$F_p = A_a \, I \, C_p \, W_p \qquad (4.11)$$

where: W_p = the weight of an element or component

C_p = coefficient given in Table 4.8

1. The values of A_a and I should be the values used for the building.

   Exceptions:

   a. For anchorage of machinery and equipment required for life-safety systems, the value of I should be taken as 1.5.

   b. For tanks and vessels containing sufficient quantities of toxic or explosive substances to be hazardous to the safety of the general public if released, the value of I should be taken as 1.5.

   c. The value of I for panel connectors should be as given in Paragraph B following.

2. The coefficient C_p is for rigid elements and components. Rigid elements are defined as those having a fixed base period less than or equal to 0.06 second. Nonrigid elements are defined as those having a fixed base period greater than 0.06 second.

3. In the absence of a detailed analysis, the value of C_p for a nonrigid component should be taken as twice the value listed in Table 4.8, but need not exceed 2.0.

4. The lateral forces determined using Equation 4.11 should be distributed in proportion to the mass distribution of the element or component.

5. Forces determined using Equation 4.11 should be used to evaluate elements or components and their connections and anchorage to the structure, and to evaluate members and connections that transfer the forces to the seismic resisting system.

6. For applicable forces in connectors for exterior panels and diaphragms, refer to Paragraphs B and C following.

7. Forces should be applied in the horizontal directions that result in the most critical loadings for design.

B. Exterior Elements. Precast or prefabricated nonbearing, nonshear wall panels or similar elements that are attached to or enclose the exterior, should be able to resist the forces per Equation 4.11 and be capable of accommodating movements of the structure resulting from lateral forces or temperature changes. Concrete panels or other similar elements should be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. Connections and panel joints should allow for a relative movement between stories of not less than (3/8) R_w times the calculated elastic story drift caused by the seismic forces, or 1/2 inch, whichever is greater.

2. Bodies of connections should have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failure at or near welds.

3. The body of the connection should be checked for a capacity of 1-1/3 times the force determined by Equation 4.11.

4. Elements connecting the bodies to the panel or the structure such as bolts, inserts, welds, dowels, etc. should have a capacity of 4 times the forces determined by Equation 4.11.

5. Elements of connections embedded in concrete should be terminated so as to effectively transfer forces to the reinforcing steel.

6. The value of the coefficient I should be 1.0 for the entire connection.

C. Diaphragms. The deflection in the plane of the diaphragm should not exceed the permissible deflection of the attached elements. A permissible deflection is that defined as the deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

1. Diaphragm Forces. Floor and roof diaphragms should be designed to resist forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (4.12)$$

(a) The force F_{px} determined from Equation 4.12 need not exceed 0.75 A_a I w_{px}, but should not be less than 0.35 A_a I w_{px}.

(b) When the diaphragm is required to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces should be added to those determined from Equation 4.12.

4.4.6 Special Procedure for Unreinforced Masonry Buildings

This is a tentative procedure based on the ATC-22 document (ATC, 1989) and the ABK Methodology (ABK, 1984). The formulas and material capacities of that methodology have been reduced to working-stress levels for consistency with the provisions of a model ordinance that has been developed in a hazard reduction program for URM buildings in California (UCBC, 1991). The special procedure is intended for buildings with flexible diaphragms and unreinforced masonry bearing and enclosure walls around the full perimeter. Other URM buildings should be analyzed by conventional procedures.

Conventional analysis procedures generally are based on lumped-mass models in which roof and floor diaphragms are considered to be relatively rigid. The weight tributary to each level is lumped with the weight of the diaphragm itself and applied to relatively flexible frames or shear walls. The special procedure of this section recognizes the special behavior of buildings with relatively flexible diaphragms and rigid shear walls. The procedure is applicable to buildings that have the following characteristics:

1. The diaphragm is a relatively flexible diaphragm of wood or untopped metal deck. The diaphragm is laterally supported by shear walls, with a diaphragm span, L.

2. The shear walls are relatively rigid. They are made of concrete or masonry. They qualify as shear walls according to the Handbook criteria for strength and stiffness. A wall is considered rigid if its total height from ground to roof is no more than 1.5 times its length.

3. There can be intervening crosswalls. Crosswalls are wood-framed walls sheathed with any of the materials described in Table 4.9. Other systems such as special moment resisting frames may be used as crosswalls provided that the yield deflection does not exceed one inch in each story height. Crosswalls must be reasonably uniformly distributed along the span of the diaphragm and shall be located no more than forty feet from a shear wall or another crosswall. Crosswalls shall have a strength of 30 percent of the diaphragm strength in the direction of consideration.

The buildings to which this procedure applies are assumed to have the following characteristic behavior:

1. The rigid shear walls do not amplify the ground motion, but simply transmit the ground motion to the ends of the diaphragm.

2. In addition to its own weight, the diaphragm receives load through its interaction with the normal walls, the walls perpendicular to the assumed direction of the earthquake load.

3. The crosswalls function as dampers to limit the motion of the diaphragm.

4.4.6.1 Special Analysis Procedure for Wood Diaphragms

This section is included to provide a recommended procedure for the evaluation of wood or untopped metal deck diaphragms in unreinforced masonry bearing wall buildings. This procedure is also recommended for use in buildings with walls composed of materials other than unreinforced masonry.

This procedure is based almost exclusively on the ABK Methodology (ABK 1984), which, in turn, is based on the results of extensive analysis and testing of wood diaphragms in unreinforced masonry buildings. These tests, performed by ABK, demonstrated that in regions of moderate or high seismicity (EPA > .10 g) the dynamic response of these diaphragms is dominated by their nonlinear hysteresis characteristics. The ABK project also showed that static analysis methods cannot predict the dynamic displacement of diaphragms subjected to moderate-to-strong levels of seismic shaking.

The recommended analysis procedure is unlike existing seismic design procedures because the use of current static analysis procedures for diaphragms does not address the two functions of diaphragms: (1) horizontal diaphragms couple the weight of the out-of-plane walls and the diaphragm weight to the end shear walls; and (2) the stiffness properties of horizontal diaphragms control the displacement of the center of the diaphragm span relative to displacements of the end shear walls.

The recommended procedure is based on allowable diaphragm span versus capacity/demand (C/D) ratios (Fig. 4.2) that were developed from data obtained by dynamic testing (ABK, 1981c) and computer modeling (ABK, 1984).

The procedure for checking the deflection of the diaphragm is as follows:

1. Calculate the C/D ratio, using the following formula:

$$C/D = 3\left(\frac{W_d}{2v\,D_d + V_{cw}}\right)\frac{0.4}{A_a} \tag{4.13}$$

where:

W_D = total dead weight tributary to diaphragm including walls perpendicular to the direction of motion

v = allowable shear values for the diaphragm (see Table 4.9)

D_d = diaphragm depth (depth of the building parallel to the direction along which analysis forces are applied)

A_a = effective peak acceleration given in Figure 3.2

V_{cw} = the capacity of the crosswalls (v from Table 4.9 for crosswalls times the length of the crosswalls)

In order to qualify, crosswalls must be reasonably well distributed along the length of the diaphragm, must have a maximum spacing of 40 feet, must have a minimum capacity equal to at least 30% of the diaphragm capacity, and must be suitably connected to the diaphragm above and below. If crosswalls are not present, $V_{cw} = 0$.

Note that in formula 4.13, the demand on the diaphragm is W_d, the capacity is $2vD_d + V_{cw}$, $A_a/0.4$ is a zone reduction factor, and the 1/3 factor reduces the demand to allowable-stress level.

If openings occur in the diaphragms adjacent to a shear wall, a revised depth, D_1, must be used. If the opening occurs in the remainder of the diaphragm, the full depth D_d is used in the analysis.

For the special case of an "open-front" building, the allowable span length for diaphragms with shear walls at the diaphragm ends may also be used. The equivalent span length L_1 and the corresponding quantity C/D ratio should be calculated as follows:

$$L_1 = 2\left(\frac{W_w}{W_d} + 1\right) L \tag{4.14}$$

$$C/D = 3\left(\frac{V_u D_d + V_{cw}}{W_d + W_w}\right) \frac{4}{A_a} \tag{4.15}$$

where W_w is the wall weight at the open end and the other variables are as defined previously. Using the C/D ratio calculated from formula 4.15, the allowable span length can be determined from Figure 4.2 and compared to the equivalent span length, L_1, computed from formula 4.14 to determine if the diaphragm meets the span limitations.

2. In Figure 4.2, plot the point determined by the above-calculated C/D ratio and the diaphragm span, L, measured between shear walls.

    a. If the plotted point falls to the left of the curve, the expected diaphragm deflection is excessive. The deficiency is in the strength of the diaphragm or the extent of qualifying crosswalls or both.

    b. If the plotted point falls to the right of the curve, the expected diaphragm deflection is acceptable, and the location of the point (whether it is in region 1, 2, or 3) determines the criteria to be used in checking the stability of the URM walls.

4.4.6.2 Stability of URM Walls

Stability of the URM walls is governed by the stiffness of the diaphragm and the stiffness of the normal walls (the URM walls perpendicular to the assumed direction of the earthquake load). If the diaphragm deflection is acceptable, as determined in the procedure above, the next step is to check the h/t ratios of the normal walls. The limiting h/t ratios are given in Table 4.12, one set of ratios for buildings "with crosswalls" and one set for "all others."

The interior walls that are parallel to the assumed direction of the earthquake load and are located between the shear walls, must meet the following criteria in order to qualify as "crosswalls."

1. The wall construction conforms to the types listed in Table 4.9.

2. The walls extend from the roof to the ground.

3. The walls are reasonably symmetrically placed along the length of the diaphragm and are spaced no more than 40 feet apart.

4. The walls at a given level have a minimum capacity of 10% of the diaphragm capacity at that level, i.e., the sum of the strengths of the qualifying walls is at least 0.30 x v x D_d.

Referring back to the check of diaphragm stiffness, acceptable diaphragms have points on Figure 4.2 that fall to the right of the curve.

If the point is in region 1: if there are qualifying crosswalls, use the limiting h/t ratios for "with crosswalls"; if the interior walls do not qualify as crosswalls use the limiting ratios for "all others."

If the point is in region 2: the limiting ratios for "with crosswalls" may be used whether or not there are crosswalls.

If the point is in region 3: the limiting ratios for "all others" must be used even if there are qualifying crosswalls.

Calculate the h/t ratios of the URM walls and check against the appropriate limiting ratios determined above. Any wall whose h/t ratio exceeds the limit is unacceptable. The deficiency is in the thickness of the wall.

4.4.6.3 Wall Anchorage-Diaphragm to Shear Wall

The diaphragm end shear should be developed into the shear wall by anchors capable of developing the force

$$F_p = C_p \times (W_d/2) \times (A_a/0.4) \tag{4.16}$$

where C_p is a response factor from Table 4.10.

The required anchorage force need not exceed the allowable shear, v x D_d, in the diaphragm. The anchors may be used at allowable loads given in Table 4.11. For existing anchors, see the testing requirements presented in Appendix E.

4.4.6.4 Wall Anchorage-Diaphragm to End Walls

The wall anchor system should be capable of developing the force

$$F_p = 0.30\ W_n \times (A_a/0.4) \qquad (4.17)$$

where W_n is the tributary weight of the wall from the mid-height of the story above to the mid-height of the story below. The anchors may be used at the allowable loads given in Table 4.11.

4.4.6.5 Strength of Shear Walls

The shear wall should first be checked by the Quick Check procedures for shear walls given in Statement 10.1.5.1. If the wall fails that check, it should be given a detailed analysis by conventional procedures, with story forces calculated as the smaller of the following:

$$F = 0.10 \times (W_n + W_d/2) \times (A_a/0.4) \qquad (4.18)$$

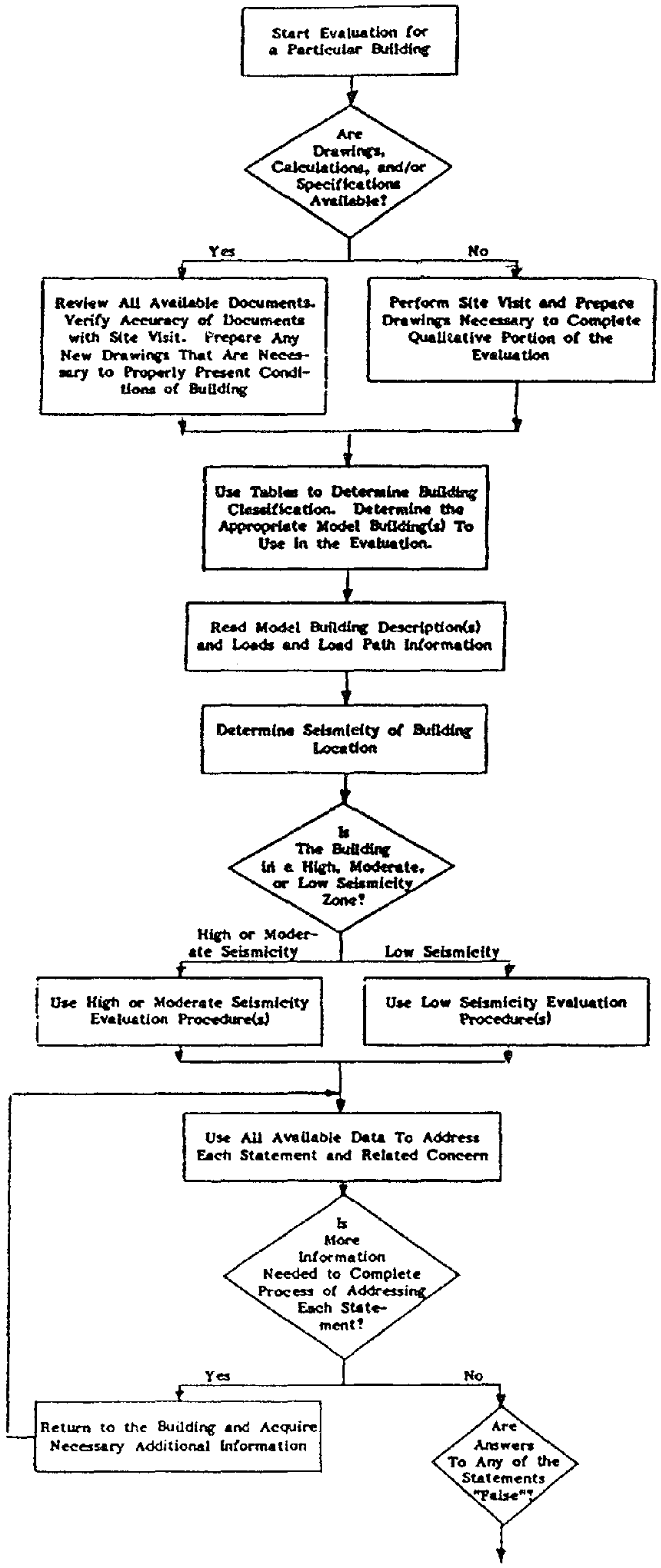
$$F = 0.10 \times (W_n + v_u/2) \times (A_a/0.4) \qquad (4.19)$$

The masonry may be used at the following allowable stresses:

Shear: as determined by the procedures of Appendix E. Existing unreinforced masonry shall not be used if the shearing stress obtained by these procedures is less than 3 psi.

Compression: 100 psi

Tension: not permitted



Flow Chart Depicting the Steps to be Followed in Using the Methodology.
(Continued on Next Page)

FIGURE 4.1

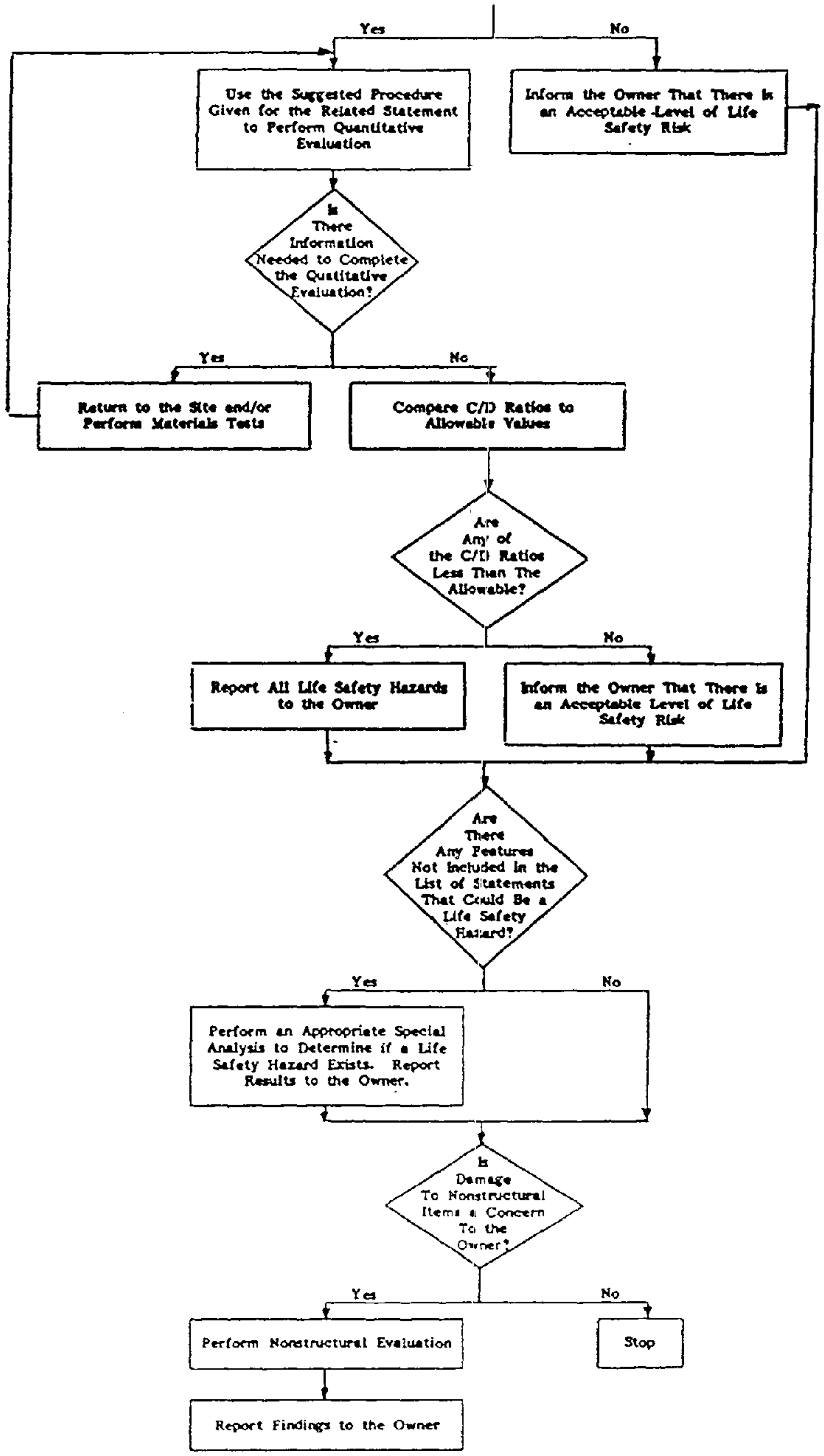
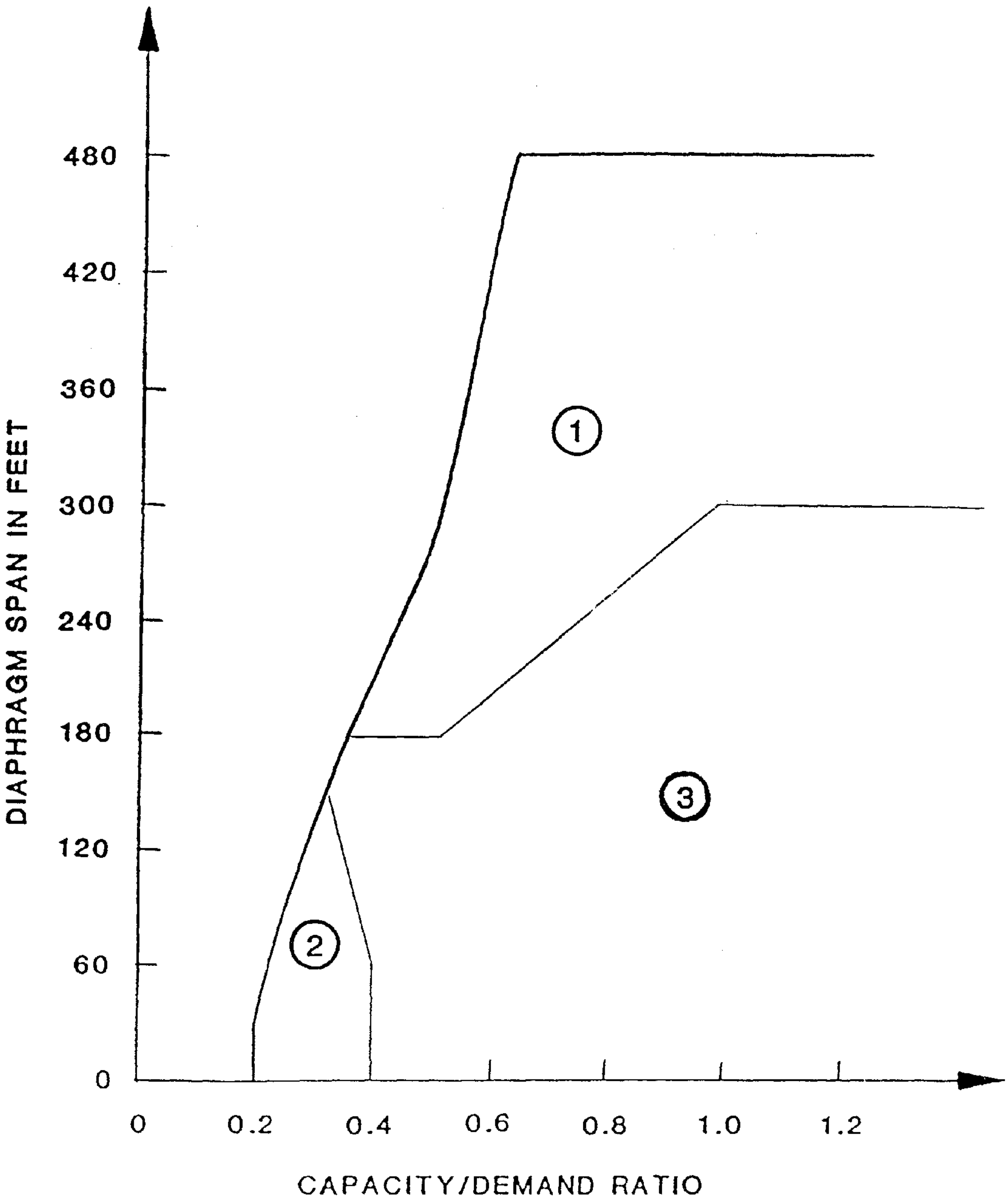


FIGURE 4.1 (CONTINUED)



Acceptable Span Lengths for Diaphragms as a Function of Calculated C/D Ratio
for Buildings in Regions of Moderate or High Seismicity
(Based on Displacement Control Concepts; from ABK (1984)).

FIGURE 4.2

TABLE 4.2

Model Buildings

I. Wood Buildings

Building 1: Wood A, Wood Frame Dwellings and Light Frames (W1)

Building 2: Wood B, Commercial or Industrial Wood Structures (W2)

II. Steel Buildings

Building 1: Steel Moment Resisting Frame Buildings (S1)

Building 2: Braced Steel Frame Buildings (S2)

Building 3: Light Moment Frame Buildings with Longitudinal Tension Only Bracing (S3)

Building 4: Steel Frame Buildings with Cast-in-Place Concrete Shear Walls (S4)

Building 5: Steel Frame Buildings with Infilled Walls of Unreinforced Masonry (S5)

III. Cast-in-Place Reinforced Concrete Buildings

Building 1: Reinforced Concrete Moment Resisting Frame Buildings (C1)

Building 2: Shear Walls Buildings (C2)

Building 3: Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry (C3)

IV. Buildings with Precast Concrete Elements

Building 1: Tilt-up Buildings with Precast Bearing Walls (PC1)

Building 2: Buildings with Precast Concrete Frames and Concrete Shear Walls (PC2)

V. Reinforced Masonry Buildings

Building 1: Reinforced Masonry Bearing Wall - Wood or Metal Deck Diaphragm Buildings (RM1)

Building 2: Reinforced Masonry - Precast Concrete Diaphragm Buildings (RM2)

VI. Unreinforced Masonry Buildings

Building 1: Unreinforced Masonry Bearing Wall Buildings (URM)

Note: The designations given at the end of each building title are used in Table 4.3.

TABLE 4.3

Matrix of Possible Building Combinations

Wall and/or Frame Elements \ Slab or Diaphragm Elements	Timber Sheathed Diaphragms with Short Spans[1]	Timber Sheathed Diaphragms with Long Spans[2]	Metal Deck with Cast-in-Place Concrete Fill[3]	Metal Deck Without Fill[4]	Cast-in-Place Concrete Slabs[5]	Precast Concrete Slabs[6]
1. Stud bearing and non-bearing walls with studs at 16" and 24" centers. May also have wood posts or steel pipe columns.	W1[7]					
2. Wood posts or steel pipe columns. Exterior walls may have rod braces or be sheathed with plywood.		W2				
3. Steel columns, tension only rod bracing, with knee bracing or rigid frame action in transverse direction.				S3		
4. Steel beams and columns. Moment or braced steel frame to resist lateral loads. Usually have nonstructural walls such as lightweight curtain, transite, or infill masonry.			S1,S2	S1,S2	S1,S2	
5. Light metal stud walls that may or may not be bearing. Exterior walls may be sheathed with stucco.			W1			
6. Complete steel frame. Lateral loads resisted by moment frame that may only include perimeter columns.			S1[7]		S1	S1,PC2

TABLE 4.3 (CONTINUED)

Wall and/or Frame Elements \ Slab or Diaphragm Elements	Timber Sheathed Diaphragms with Short Spans[1]	Timber Sheathed Diaphragms with Long Spans[2]	Metal Deck with Cast-in-Place Concrete Fill[3]	Metal Deck Without Fill[4]	Cast-in-Place Concrete Slabs[5]	Precast Concrete Slabs[6]
7. Complete steel frame. Lateral loads resisted by braced frame action. Frame connections are not moment resisting.			S2	S2	S2	S2,PC2
8. Complete steel frame. Non-moment resisting, with CIP concrete shear walls to resist lateral loads.			S4	S4	S4	S4,PC2
9. Complete, non-moment resisting steel frame with precast concrete shear walls.				S4,PC2	S4,PC2	
10. Incomplete steel frame with concrete core shear walls.			S4	S4	S4	S4,PC2
11. Complete steel frame dual system with bracing and moment frame—all steel.			S1,S2		S1,S2	
12. Complete steel frame dual system with concrete shear walls.			S4		S4	
13. Complete ductile concrete moment resisting frame—CIP.			C1[7]	C1	C1	C1,PC2
14. CIP concrete non- or semi-ductile moment resisting frame.			C1	C1	C1	C1,PC2
15. Squat CIP concrete shear walls with CIP concrete frame.			C2	C2	C2	C2,PC2

TABLE 4.3 (CONTINUED)

Wall and/or Frame Elements \ Slab or Diaphragm Elements	Timber Sheathed Diaphragms with Short Spans[1]	Timber Sheathed Diaphragms with Long Spans[2]	Metal Deck with Cast-in-Place Concrete Fill[3]	Metal Deck Without Fill[4]	Cast-in-Place Concrete Slabs[5]	Precast Concrete Slabs[6]
16. CIP tall concrete shear walls with CIP frame.			C2	C2	C2	C2,PC2
17. Precast shear walls with or without precast frame.						PC2
18. Complete CIP ductile moment frame with CIP shear walls—dual system.					C1,C2,	C1,C2, PC2
19. Incomplete ductile steel or CIP concrete moment resisting frame with CIP concrete core shearwalls.			S1 or C1, C2	S1 or C1, C2	S1 or C1, C2	S1 or C1, C2, PC2
20. Incomplete non-ductile moment resisting CIP concrete or steel frame with CIP concrete core shear walls.			C1,C2 or S4	C1,C2 or S4	C1,C2 or S4	C1,C2 or PC2,S4
21. Unreinforced brick masonry walls.	URM[7]		URM or S5	URM or S5	URM or S5 or C3	URM or S5 or C3,PC2
22. Unreinforced concrete brick masonry walls.	URM		URM or S5	URM or S5	URM or S5 or C3	URM or S5 or C3,PC2
23. Reinforced brick masonry walls	RM1		RM1	RM1		RM2
24. Reinforced concrete block masonry walls. May not grout all cells.	RM1		RM1	RM1		RM2

TABLE 4.3 (CONTINUED)

Wall and/or Frame Elements \ Slab or Diaphragm Elements	Timber Sheathed Diaphragms with Short Spans[1]	Timber Sheathed Diaphragms with Long Spans[2]	Metal Deck with Cast-in-Place Concrete Fill[3]	Metal Deck Without Fill[4]	Cast-in-Place Concrete Slabs[5]	Precast Concrete Slabs[6]
25. Tilt-up reinforced concrete bearing walls.	PC1		PC1	PC1		PC1,PC2
26. CIP reinforced concrete bearing walls.	W2,C2		C2	C2		PC2,C2
27. Multiple story, reinforced masonry bearing wall construction.					RM1, C2	RM2
28. Concrete parking structure below wood frame structure.	W1,C2					
29. Exterior Walls of adobe or rock.	URM	URM				

Footnotes:

[1] Timber sheathed—straight or diagonal, or plywood, supported by wood purlins or rafters, usually with short spans.
[2] Timber sheathed—straight or diagonal, or plywood, supported by glulams, wood trusses or steel beams.
[3] Metal deck with cast-in-place reinforced concrete fill, supported by steel beams, girders, or open web joists.
[4] Metal deck without concrete fill, supported by steel beams, girders, or open web joists.
[5] Cast-in-place concrete, one-way joists, two-way waffle slab or two-way slabs; does not include flat slabs.
[6] Precast concrete slabs, with or without cast-in-place concrete topping slab.
[7] See Table 4.2 for model building designation.

TABLE 4.4

R_w Factors for Various Structural Systems

Chapter/ Section	Building Type or Description	R_w
5	Wood-Frame Buildings	
	a. Bearing Wall System with plywood walls; three stories or less	8
	b. Bearing Wall System with walls of other materials or plywood walls greater than three stories	6
	c. Building Frame System with plywood walls; three stories or less	9
	d. Building Frame System with walls of other materials or plywood walls greater than three stories	7
6.1	Steel Moment Frame Buildings	
	a. Moment resisting space frame provided with ductile details	12
	b. Moment resisting space frame not provided with ductile details	6
6.2	Braced Steel Frame Buildings	
	a. Braced frames where bracing carries gravity loads	6
	b. Concentric Braced Frame in Building Frame System	8
	c. Braced frame in dual system with ductile moment resisting space frame	10
6.3	Light Steel Moment Frame Buildings with Longitudinal Tension Only Bracing	
	a. Light steel frame bearing walls with tension only bracing	4
6.4	Steel Frame Building with Cast-in-Place Concrete Shear Walls	
	a. Bearing Wall System	6
	b. Building Frame System	8
	c. Dual System with ductile moment resisting space frame	12
6.5	Steel Frame Building with Infill Walls of Unreinforced Masonry	6
7.1	Concrete Moment Resisting Frame Buildings	
	a. Moment resisting frame provided with ductile details	12
	b. Moment resisting frame not provided with ductile details	5

TABLE 4.4 (CONTINUED)

Chapter/ Section	Building Type or Description	R_W
7.2	Cast-in-Place Concrete Shear Wall Buildings	
	a. Bearing Wall System	6
	b. Building Frame System	8
	c. Dual System with concrete moment resisting frame	9
7.3	Concrete Frame Building with Infilled Walls of Unreinforced Masonry	5
8.1	Tilt-up Buildings with Precast Bearing Wall Panels	6
8.2	Precast Concrete Frame and Concrete Shear Wall Buildings	
	a. Bearing Wall System	6
	b. Building Frame System	8
	c. Dual system with precast concrete moment resisting frame	9
9.1	Reinforced Masonry Bearing Wall Buildings with Diaphragms of Wood or Metal Deck with or without Concrete Fill	
	a. Bearing Wall System	6
	b. Building Frame System	8
9.2	Reinforced Masonry Bearing Wall Buildings with Precast Concrete Diaphragms	
	a. Bearing Wall System	6
	b. Building Frame System	8
10	Unreinforced Masonry Bearing Wall Buildings	3

TABLE 4.5

Site Coefficients

Type	Description	S Factor
S_1	A soil profile with either: (a) rock-like material characterized by shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 200 feet or more.	1.2
S_3	A soil profile that contains 30 feet or more of either soft to firm clays or loose sands.	1.5
S_4	A soil profile containing more than 40 feet of soft clay.	2.0

The site factor shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, Soil Profile S_3 shall be used unless there is reason to believe Soil Profile S_4 may be present at the site, in which case Soil Profile S_4, should be used.

Soil profiles shall be rated at the base of the building, not at the end of piles, caissons, or other foundation elements.

TABLE 4.6

Occupancy Categories

Occupancy Categories		Occupancy Type of Function of Structure
I	Essential Facilities	Hospitals and other medical facilities having surgery and emergency treatment areas
		Fire and police stations
		Tanks or other structures containing housing, supporting water or other fire-suppression materials, or equipment required for the protection of essential or hazardous facilities, or special occupancy structures
		Emergency vehicle shelter and garages
		Structures and equipment in emergency preparedness centers
		Stand-by power-generating equipment for essential facilities
		Structures and equipment in communication centers and other facilities required for emergency response
II	Hazardous Facilities	Structures housing, supporting, or containing sufficient quantities of toxic or explosive substances to be dangerous to the safety of the general public if released
III	Special Occupancy Structures	Covered structures whose primary occupancy capacity > 300 persons
		Buildings for schools (secondary and below) or day-care centers; capacity > 500 students
		Medical facilities with 50 or more resident incapacitated patients, but not included above
		Jails and detention facilities
		All structures with occupancy > 5000 persons
		Structures and equipment in power-generating stations and other public utility facilities not included above, and required for continued operation
IV	Standard Occupancy Structures	All structures having occupancies or functions not listed above.

TABLE 4.7

Occupancy Category Importance Factors

	Occupancy Category[1]	Importance Factor I
I.	Essential Facilities	1.0[2]
II.	Hazardous Facilities	1.0[2]
III.	Special Occupancy Structures	1.0
IV.	Standard Occupancy Structures	1.0

Footnotes:

[1] Occupancy types or functions of structures within each category are listed in Table 4.6.

[2] An I value of 1.0 is used in this document because life safety rather than damage control is the primary requirement. An I of 1.25 may be used for these facilities if greater protection against damage to both structural and nonstructural elements is desired.

TABLE 4.8

Horizontal Force Factor C_p Applicable to Rigid Items[1]

Elements of Structures and Non-Structural Components	Value of C_p
I. Part or Portion of Structure	
1. Walls, including the following:	
a. Unbraced (cantilevered) parapets	2.0
b. Other exterior walls above the ground floor	0.75
c. All interior bearing and nonbearing walls and partitions	0.75
2. Penthouses (except where framed by an extension of the building frame)	0.75
3. Connections for prefabricated structural element other than walls, with force applied at center of gravity[2]	0.75
4. Diaphragms[3]	
II. Nonstructural Components	
1. Exterior and interior ornamentations and appendages	2.0
2. Chimneys and other stacks	
a. Supported on or projecting as an unbraced cantilever above the roof more than one-half its total height	2.0
b. All others, including those supported below the roof with unbraced projection above the roof less than one-half its height, or braced or guyed to the structural frame at or above its center of mass	0.75
3. Signs and billboards	2.0.
4. Mechanical and electrical equipment and machinery[4]	0.75
5. Tanks and vessels (plus contents) including support systems and anchorage	0.75
6. Storage racks (including contents)	0.75
7. Anchorage for permanent floor-supported cabinets and bookstacks more than 5 feet in height (includes contents)	0.75

TABLE 4.8 (CONTINUED)

Elements of Structures and Non-Structural Components		Value of C_p
8.	Anchorage for suspended ceilings and light fixtures[5]	0.75

Footnotes:

[1] See Section 4.4.5.A.2 for the definition of rigid.
[2] These forces should be resisted by positive anchorage and not by friction.
[3] See Section 4.4.5.C.
[4] Equipment and machinery should include but not be limited to such items as boilers, heat exchangers, chillers, pumps, motors, air-handling units, cooling towers, transformers, switch gear, and control panels. It should also include the major piping, ducting, conduit, cable trays, etc., which serve such equipment and machinery.
[5] Ceiling weight should include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the lateral seismic force, a ceiling weight of not less than four psf should be used.

Table 4.9 Allowable Values, v, of Existing Construction

This table is for use only in the special procedure of Section 4.4.6.

Description of Existing Construction	Allowable values, v, in lb/ft for seismic shear
1. Horizontal Diaphragms	
a. Roofs with straight sheathing and roofing applied directly to the sheathing	100
b. Roofs with diagonal sheathing and roofing applied directly to the sheathing	250
c. Floors with straight tongue-and-groove sheathing	100
d. Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular	500
e. Floors with diagonal sheathing and finished wood flooring	600
2. Crosswalls	
a. Plaster on wood or metal lath, allowable for each side	200
b. Plaster on gypsum lath	175
c. Gypsum wall board, unblocked edges	75

Table 4.10 Response Factor, C_p, for Shear Connection of Horizontal Diaphragm to Shear Walls

This table is for use only in the special procedure of Section 4.4.6.

<u>Description of Existing Construction</u>	<u>C_p</u>
Roofs with straight or diagonal sheathing and roofing applied directly to the sheathing, or floors with straight tongue-and-groove sheathing	0.2
Double or multiple layers of boards with edges offset and blocked plywood systems	0.3

Table 4.11 Allowable Values for Wall Connections

This table is for use only in the special procedure of Section 4.4.6.

Element	Allowable Value
1. Shear Bolts	
Shear bolts embedded a minimum of 8 inches into unreinforced masonry walls. Bolt centered in a 2-1/2-inch-diameter hole with dry-pack or non-shrink grout around circumference of bolt.[1,3]	100% of the values for plain masonry specified for solid masonry in the Building Code. No values larger than those given for 3/4 inch bolts shall be used.
2. Tension Bolts	
Tension bolts and tension dowels extending entirely through unreinforced masonry walls secured with bearing plates on far side of 3 wythe minimum wall with at least 30 square inches of area.[2,3]	1800 lbs per bolt 900 lbs for 2 wythe walls
3. Wall Anchors	
a. Bolts extending to the exterior face of the wall with a 2-1/2-inch round plate under the head, installed as specified for shear bolts, and spaced not closer than 12 inches on centers.[1,2,3]	600 lbs per bolt
b. Bolts extending to the exterior face of the wall with a 2-1/2-inch round plate under the head and drilled at an angle of 22-1/2 degrees to the horizontal, installed as specified for shear bolts.[1,2,3]	1200 lbs per bolt

[1] Bolts to be tested as specified in Appendix B.

[2] Bolts to be 1/2-inch minimum in diameter.

[3] Drilling for bolts and dowels shall be done with an electric rotary drill. Impact tools shall not be used for drilling holes or tightening anchors and shear bolt nuts.

Table 4.12 Allowable Value of Height-Thickness Ratio of Unreinforced Masonry Walls with Minimum Quality Mortar[1,2]

	Buildings with Crosswalls[2]	All Other Buildings
Walls of one-story buildings	16	13
First-story wall of multi-story buildings	16	15
Walls in top story of multi-story buildings	14	9
All other walls	16	13

[1] Minimum quality mortar shall be determined by laboratory testing in accordance with Appendix B.

[2] Walls shall qualify as defined in Section 4.4.6.

SECTION 5

SEISMIC EVALUATION OF WOOD-FRAME BUILDINGS

5.1 Building Descriptions

5.1.1 Type 1 Buildings - Dwellings

Type 1 buildings are typically single or multiple family units one or more stories in height. These buildings usually include subfloors of straight or diagonal sheathing or plywood topped by finished floors. Tongue and groove plank floors may also be used. The roof diaphragms may consist of straight or diagonal sheathing, or plywood. Floor framing typically consists of 2x construction at 16 or 24 inch spacing that spans to stud walls or larger beams that are in turn supported by wood posts or steel pipe columns. The foundations are generally concrete, although older construction often employed brick. Plates of stud walls are often not bolted to the foundation in older construction. In some dwellings, the first floor is supported above the crawl space by 2x construction that spans to beams and 4x4 posts. These posts may not be well connected to the structural components on which they rest. Stud bearing and nonbearing walls typically have studs at 16 or 24 inch centers. Lateral resistance is provided by plywood, sheetrock, plaster, exterior siding, let-in braces, etc. Rooms are generally 24 feet or less in each direction. Ceilings and partitions may be finished with plaster, sheetrock, panel board, or tile. Dwellings often have flues or chimneys that may be composed of brick, wood, and stucco using tile or patented flues. Chimneys may or may not be tied into other construction. The exterior of these buildings may be fully or partially covered with veneer.

5.1.2 Type 2 Buildings - Commercial or Industrial Structures

Type 2 buildings usually are 5,000 square feet or larger, with few, if any, interior walls. The roof diaphragms may be plywood, straight or diagonal sheathing, or planking. Straight-sheathed roofs may also have rod bracing. The roof often has large openings for skylights or HVAC equipment. The major framing elements often have longer spans than Type 1 wood-frame buildings, from a minimum of 40-50 feet to a maximum of 80-100 feet. These major elements can consist of wood or steel trusses, or glu-lam or steel beams, supported by wood posts, pipe columns, or steel columns. The foundations consist of concrete that may or may not be reinforced. The exterior walls are of stud-framed construction similar to that of Type 1 wood-frame buildings. Wall sheathing can consist of plywood, stucco, plaster, or paneling. The walls may have rod bracing. Because there are few or no interior walls, lateral loads are resisted by the exterior walls.

5.2 Performance Characteristics (Type 1 and Type 2 Buildings)

Wood-framed buildings generally do not pose a significant life-safety threat during seismic events except in rare cases. However, building contents may be badly shaken. The following statements list some specific performance characteristics that these buildings may exhibit:

1. If the stud walls are not adequately bolted to the foundation, the structure can slide off the foundation. Anchor bolts may fail if edge distance in concrete is insufficient.

2. If the stud cripple wall below the first floor is unbraced, the structure may roll off the cripple studs below the first floor and fall to the foundation.

3. If no diagonal rod bracing is provided for a straight sheathed roof, proper diaphragm action may also depend on the ceiling material. Diaphragms with large span/depth ratios may experience distress.

4. Lack of continuous and adequate collector elements can lead to distress at discontinuities, re-entrant corners or large openings, and load transfer problems between diaphragms and walls.

5. Split level floors can separate at their intersections if not properly connected.

6. Two-story construction with large openings at the first floor (e.g. garage doors) can collapse at the opening if not properly connected to a bracing element or not capable of accommodating the torsional displacements.

7. Unreinforced masonry chimneys that extend above the roof level may break off at the roof level and fall onto the structure or the property below.

8. Inadequate attachment of masonry veneer to wood framing can cause the veneer to fall.

9. Glass in large skylights of Type 2 wood-frame buildings can shatter due to diaphragm deformations.

10. Walls with large openings or a long length of clerestory window may undergo distress.

11. In multistory Type 2 buildings, parapets that are unreinforced and have large height to thickness ratios or that are not anchored to the roof diaphragm may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

12. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may tend to vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

13. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

5.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

5.3.1 Type 1 Structures

1. House with cripple walls, Long Beach 1933 (Steinbrugge, 1982, p. 126). Collapse of house.

2. Unreinforced chimneys, Santa Rosa 1969 (Steinbrugge, 1982, p. 127). Chimneys broken. MM VII-VIII.

3. Fireplace, Daly City 1957 (Steinbrugge, 1982, p. 128). Fireplace damage. MM VII.

4. Split-level house, San Fernando 1971 (Steinbrugge, 1982, p. 128). House collapsed. MM VIII-XI.

5. Residences (general), Tehachapi 1952 (SEAONC, 1952, p. 49-50). Cripple studs fell over on timber house, fell off foundations, (Steinbrugge and Moran, 1954 p. 223; CDMG, 1955, p. 218). MM VIII-IX.

6. Chimneys, Tehachapi 1952 (SEAONC, 1952, pp. 50-51). Chimneys failed.

7. House, Arvin 1952 (SEAONC, 1952, p. 131). House fell off foundations. No anchor bolts to sill. MM VIII.

8. Houses, San Fernando 1971 (Murphy, 1973, p. 698). Various house failures.

9. House opposite Hillside Apartments, Anchorage 1964 (C&GS, 1967, p. 133). House undamaged.

10. Wood houses, Santa Rosa 1969 (Steinbrugge et al., 1970, p. 9-12). Various failures. MM VII-VIII.

11. Veterans Administration Hospital, Sylmar, San Fernando 1971 (Murphy, 1973, p. 655). There were seven wood frame buildings with varying degrees of damage - some with no apparent damage. No collapses. MM VIII-XI.

12. Tehachapi Inn, Tehachapi 1952 (SEAONC, 1952, p. 23). Two-story frame, composition shingle siding, some distortion of shingle sheathing. MM VIII-IX.

13. Clark Hotel, Tehachapi 1952 (Degenkolb, 1955, pp. 1281 and 1282; SEAONC, 1952, p. 32). Board and batten timber frame, very loose. No damage. MM VIII-IX.

14. Wood Frame, One-Story Dwellings (general), Coalinga, California 1983 (CDMG, 1983). Houses moved completely off of subfloor cripple walls that were not properly anchored to the foundations. No plywood sheathing on cripple walls, so they were often inadequate for transferring shear. Jack post floor supports also failed. Collapse of wooden porch awnings due to improper anchorage. Collapsed chimneys broke at roofing or pulled away from the wall. MM VIII.

15. Oil King School, Coalinga, California 1983 (CDMG, 1983, p. 39). Plywood roof diaphragm on wood joists on steel beams. Plywood shear walls. No apparent structural damage. Minor arch damage, broken windows. MM VIII.

16. Wood Frame One-Story Dwellings (general), San Francisco 1906 (State Earthquake Investigation Commission, 1969, Part 1, pp. 220-241). Chimneys fell. Houses disturbed on their foundations. Foundations cracked. High post (wood) underpinning gave way. In some areas, soil failures caused severe leaning or complete collapse. Often row houses held each other up. MM VI-X.

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.

5.3.2 Type 2 Structures

1. Tehachapi Hay and Grain, Tehachapi 1952 (Degenkolb, 1955, p. 1284; SEAONC, 1952, p. 18). Wood frame covered with corrugated iron-not designed for earthquake. No damage. MM VIII-IX.

2. Corrugated iron warehouse, Tehachapi 1952 (Degenkolb, 1955, p. 1284; SEAONC, 1952, p. 55). Wood frame covered with corrugated iron, not designed for earthquake. No damage. MM VIII-IX.

3. Town and Country Market, Tehachapi 1952 (Degenkolb, 1955, p. 1283, CDMG, 1955, p. 264; SEAONC, 1952, p. 25). Wood frame, stucco walls, open front, wood roof with horizontal sheathing on bow-string trusses, rod-braced roof, broken windows. Open for trade next day. MM VIII-IX.

4. Tehachapi High School, Tehachapi 1952 (SEAONC, 1952, p. 9, Fig. 6). Timber classrooms designed to Field Act Regulations. No damage. MM VIII-IX.

5. Tehachapi Veteran's Hall, Tehachapi 1952 (SEAONC, 1952, p. 18). Wood and stucco, tile roof. Damaged. MM VIII-IX.

6. Elementary School, Tehachapi 1952 (SEAONC, 1952, p. 28). Timber school designed to Field Act Regulations. No damage-light fixtures fell. MM VIII-IX.

7. Church, Tehachapi 1952 (SEAONC, 1952, pp. 30-31). Frame and stucco building. No damage. MM VIII-IX.

8. Catholic Church, Tehachapi 1952 (SEAONC, 1952, p. 51). Small stucco cracks and broken windows. MM VIII-IX.

9. Southern Pacific Railroad Station, Tehachapi 1952 (Steinbrugge and Moran, 1954, p. 226). All timber. No damage. MM VIII-IX.

10. Grocery Store, Tehachapi 1952 (SEAONC, 1952, p. 57). Frame and stucco, one-story. Minor plaster cracks. MM VIII-IX.

11. Vineland School (New), Tehachapi 1952 (SEAONC, 1952, p. 135). All timber, designed under Field Act. No damage. MM VIII.

12. Van Gough Elementary School, San Fernando 1971 (Murphy, 1973, p. 675). Field Act timber school subject to site movements shifted; no collapse. Site and school repaired for $144,000. MM VIII-XI.

13. Hubbard Street Elementary School, San Fernando 1971 (Murphy, 1973, p. 675). Several timber one-story buildings. Ground breakage near site, bungalows shifted up to 6 inches. Repairs to buildings and site, $37,600. MM VIII-XI.

14. Fenton Avenue Elementary School, San Fernando 1971 (Murphy, 1973, p. 676). School timber buildings. Total repairs, $4,600. MM VIII-XI.

15. Sylmar High School, San Fernando 1971 (Murphy, 1973, p. 676). Several buildings, gym had steel trusses; rest had timber trusses. Auditorium had folded prestressed concrete plate roof; steel in bleachers. This was the most damaged Field Act school in history. MM VIII-XI.

16. Harding School, San Fernando 1971 (Murphy, 1973, p. 680). Timber classrooms, multipurpose room, one-story wood frame. Reoccupied one week after earthquake except for two classrooms. Floor cracks 2 inches wide, with 3 inches vertical displacement. Total repair costs, $27,000. MM VIII-XI.

17. Sylmar Elementary School, San Fernando 1971 (Murphy, 1973, p. 680). One-story wood, ground cracking, light fixture fell, equipment moved. Portable building moved considerably. $25,000 total repair. MM VIII-XI.

18. Coalinga Junior High School, Coalinga, California 1983 (CDMG, 1983, pp. 41-42). Library had diagonal roof sheathing supported by wood joists and steel beams. North-South lateral loads are resisted by sheathed end walls. Steel frames resist east-west forces. No bracing was provided in the north wall. Damage included broken glass in north and south wall elevations. Torsion occurred due to lack of north wall bracing. MM VIII.

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 (Stockton, 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 Loads and Load Paths

5.4.1 Type 1 Buildings

Gravity loads are transferred from floor and roof sheathing to joists that span between stud walls or larger beams. The interior wood posts that support these elements are typically founded on individual concrete footings.

Lateral loads are transferred from the diaphragm sheathing to both exterior and interior walls through collector elements. The walls transfer the forces to the foundation elements usually through bolts to stud-wall plates.

Typical floor dead weights may range between 10 and 25 psf. Live loads may be as high as 40 psf for typical occupancies.

5.4.2 Type 2 Buildings

Gravity loads are transferred from the roof sheathing to joists and then to major framing elements such as wood or steel trusses or glu-lam or steel beams. These elements typically span in the transverse direction to the exterior wall elements. Wood posts, steel columns, or pipe columns may also support these major elements. Concrete foundations support the exterior walls and columns.

Lateral loads are transferred from the roof diaphragm to the exterior walls and down into the concrete foundations. Similar action occurs in both the longitudinal and transverse directions.

Typical roof dead loads vary from 10 to 25 psf depending on the roof materials. Roof live loads are typically taken as 20 psf before allowable reductions for large unsupported areas.

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 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.

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 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.

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 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 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

SECTION 6

SEISMIC EVALUATION OF STEEL FRAMED BUILDINGS

6.1 Seismic Evaluation of Steel Moment Resisting Frame Buildings

6.1.1 Building Description

This building includes floor and roof diaphragms that are generally composed of either metal decking with concrete fill or cast-in-place concrete slabs. Major floor framing may consist of steel beams or girders, or open web joists. Steel columns typically comprise the vertical structural elements. Exterior walls may be either metal or precast concrete panels, or brick masonry.

6.1.2 Performance Characteristics

Steel moment resisting frame buildings are typically more flexible than structural systems with shear walls or braced frames. This low stiffness can result in large interstory drifts that may lead to extensive nonstructural damage. The following statements list some specific performance characteristics that these buildings may exhibit:

1. Low frame stiffness can lead to excessive drift that can cause damage to non-structural items such as partitions, ceilings, lights, windows, etc. Such drifts can lead to P-Δ excessive stresses in the main lateral force resisting elements. See Section 11 (Non-structural elements) for more detail.

2. Exterior masonry veneer or precast wall panels can fall if their connections to the building frames have insufficient strength, displacement capacity, and/or ductility. Panel elements can also fail if they cannot accommodate the interstory drift. Panels with insufficient joint size can work against each other.

3. Inelastic action at beam-column joints can cause residual displacements and/or failure of girder-flange-to-column connections.

4. If glazing is not sufficiently isolated from structural actions, it can fail and fall out onto the adjacent property.

5. Insufficient isolation of nonstructural masonry walls from the structural system may stiffen the building and alter the assumed structural response. If these infilled walls are not properly detailed, they may fail due to the lateral forces.

6. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent buildings are not at the same elevation, and where there are no shear walls parallel to the pounding forces located to directly resist these loads (i.e., at the end of the structure adjacent to the other building).

7. Frame connections, including column splices, can fail if not detailed to develop the member capacities. Bolted and partial penetration welded connections may not be capable of developing these capacities.

8. Strong beam-weak column frames can lead to hinge formation in the columns that may cause instabilities in the gravity load carrying system.

9. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing have not been provided.

10. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities may concentrate damage in the "soft" stories.

6.1.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Bunker Hill Tower, San Fernando 1971 (Murphy, 1973, p. 423). Thirty-two-story moment resisting steel frame, 1968 offices and apartments, 26 miles south of epicenter. Perimeter moment frame, box columns with (tube action) welded connections. Designed for 1967 L.A. Building Code, with 5 inches lightweight concrete slab on steel beams and girders. Columns were fireproofed with gypsum wallboard. Gypsum partitions, glass curtain wall, peak first floor acceleration 14 percent g sustained minor damage (three windows, a few partitions).

2. KB Valley Center, San Fernando 1971 (Murphy, 1973, p. 449). A 16-story office tower with perimeter steel frame lateral force resisting system. There was no observed damage to the structural system and only minor nonstructural damage. The building was designed in 1969 for the L.A. Code and every effort was made to minimize torsional forces in the structure. MMI VII.

3. Kajima International Building, San Fernando 1971 (Murphy, 1973, p. 509) A 15-story office building with a complete steel moment resisting space frame in each direction. There was no observed damage to the structural system and nonstructural damage was minimal and isolated to the plaster partitions around the elevator and stair shafts and broken glass. MMI VII

4. Union Bank Square, San Fernando 1971 (Murphy 1973, p. 575) A 42-story office building with complete moment resisting steel frame. Damage was limited to plaster cracking in cove and stair walls, floor tile damage, and rupture of the seismic separation joints. MMI VII.

5. 1901 Avenue of the Stars, San Fernando 1971 (Murphy, 1973, p. 597). Nineteen-story office building above ground, four parking levels below, 24 miles south of epicenter, 15 percent g measured peak acceleration, $14,000 building damage to repair glass and paint damage, four complete ductile moment frames in longitudinal direction, 5 X-braced frames in transverse direction. MM VI-VII.

6. Coalinga High School, Agricultural Facility, Coalinga, California 1983 (CDMG, 1983, pp. 42-46). Steel frame building. No structural damage noted. Some non-structural damage-roof vents displaced, ceiling grid displaced, suspended light fixtures pulled loose. Unanchored water heater leg buckled, broken overhead gas line. MM VIII.

7. Santa Clara County Office Building, San Jose, 1984 (CDMG, 1984, pp. 91-150) Moment steel frame building. No structural damage. Strong motion records indicate maximum response after main event with significant motion continuing 60 seconds after main event. Significant disruption to contents.

6.1.4 Loads and Load Paths

Gravity loads are transferred from the floor slabs to floor framing elements composed of steel beams or open web joists. Floor girders are supported by steel columns that transfer loads to the foundation.

Lateral loads are transferred from the floor diaphragms to the moment resisting frames. Moment frame action between the steel girders and columns is produced by full or partial moment connections. The moment frames may include all columns, or only those on the building perimeter.

Typical floor dead weights may range between 70 and 110 pounds per square foot (psf). Live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy.

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.

<u>6.1.5.1 Evaluation of Materials</u>

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

> <u>Concern:</u> 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.
>
> <u>Procedure</u>: 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.

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

> <u>Concern:</u> 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.
>
> <u>Procedure:</u> 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.
>
> <u>Recommended C/D Ratio</u>: 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 0.4 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.

6.2 Seismic Evaluation of Braced Steel Frame Buildings

6.2.1 Building Description

This building includes floor and roof diaphragms that are generally composed of either metal decking with concrete fill or cast-in-place concrete slabs. Major floor framing may consist of steel beams or girders, or open web joists. Steel columns typically comprise the vertical structural elements. Lateral loads are carried by vertical truss action of steel beams, columns and diagonal braces. Exterior walls may be either metal or precast concrete panels. In older buildings, the exterior may be composed of masonry or concrete, with an architectural facing.

6.2.2 Performance Characteristics

These buildings are typically stiffer than steel moment resisting frame structures, so interstory drifts and the resulting nonstructural damage should be reduced. Damage to bracing elements may reduce the stiffness and increase the story drifts. The following statements list some specific performance characteristics that these buildings may exhibit:

1. Large seismic events can cause buckling and/or tension yielding of the diagonal braces or failure of bolted connections at the net section, which may lead to a reduction of their load carrying capacity and stiffness, increasing the building drifts (and, therefore, nonstructural damage) significantly. Damage to other structural elements may occur if there is not sufficient moment capacity in the frame.

2. Exterior precast wall panels can fall if their connections to the building frames have insufficient strength, displacement capacity, and/or ductility. Panel elements can also fall if they cannot accommodate the interstory drift. Panels with insufficient joint size can work against each other.

3. Frames with Chevron, "V," or "K" bracing may have smaller capacities than similar braced frames with different configurations.

4. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent buildings are not at the same elevation, and where there are no shear walls parallel to the pounding forces located to directly resist these loads (i.e., at the end of the structure adjacent to the other building).

5. Parapets that are unreinforced and have large height-to-thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

6. Buildings with substantial plan irregularities, such as T, L, U or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e. re-entrant corners) if separation joints or special reinforcing has not been provided.

7. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities may concentrate damage in the "soft" stories.

6.2.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. First Federal Savings, Anchorage 1964 (C&GS, 1967, p. 175). Three-story complete simple steel frame, metal deck and concrete floors, steel X-bracing in one wall, reinforced concrete block walls (not bearing and not shear), block not reinforced according to drawings. Designed for Zone 3 of UBC. Steel framing was essentially undamaged, but block walls generally failed.

2. Foothill Medical Center, San Fernando 1971 (Murphy, 1973, p. 179). Two-story, 65 ft x 200 ft simple steel frame, open web joists, 3-1/2 inch lightweight concrete slab, X-braced. Braces not fabricated according to design. Bent members and much non-structural damage, with a damage ratio estimated at 10 percent. Designed for 1962 Los Angeles Building Code. MM VIII-XI.

3. 1901 Avenue of the Stars, San Fernando 1971 (Murphy, 1973, p. 597). Five braced frames in transverse direction, all columns ductile moment frame in longitudinal direction.

4. Call Building, San Francisco 1906 (USGS, 1907, pp. 34, 81-83). Fifteen stories plus dome. Steel braced frame. Twenty-five-foot deep foundation consisting of grillage of steel beams embedded in concrete. Knee braces at column girder joints. Three-inch thick hollow tile partitions. Reinforced cinder concrete floors. Furred ceilings of wire lathing and light furring strips. Some bending and stretching of diagonal braces. Suspended ceilings destroyed because of lack of proper fastenings. Granite and sandstone curtain walls were not damaged by earthquake. MM VII.

5. Ferry Building, San Francisco 1906 (State Earthquake Investigation Commission, 1969, Part 1, p. 235; EERI, 1973b, pp. 105-108). Steel braced frame with deep piling and grillage foundations. Sheared rivets, rods stretched beyond elastic limit in tower. Cracks in brick walls. Connection detailing not considered to be ductile. Portions of masonry walls fell. Building located in area of severe shaking but withstood earthquake. MM IX.

6.2.4 Loads and Load Paths

Gravity loads are transferred from the floor slabs to the floor framing elements composed of steel beams or open web joists. Floor girders are supported by steel columns that transfer loads to the foundation.

Lateral loads are transferred from the floor diaphragms to collector elements and to the braced frames. Vertical truss action of the beams, columns and diagonals transfer these forces through axial stresses to the foundation. Simple connections are often used at the braced frame connections. The building may or may not have a complete gravity load resisting moment frame as a secondary lateral force resisting system.

Typical floor dead weights may range between 70 and 110 psf. Live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy.

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 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 6.2.5.6: 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 0.4 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.

6.3 Seismic Evaluation of Light Steel Moment Frame Buildings with Longitudinal Tension-Only Bracing

6.3.1 Building Description

This building is typically a one-story industrial structure. Plan dimensions vary greatly, although the width of many of these buildings are small enough so that no interior columns are required. The roof is typically metal decking without concrete fill and is supported by steel purlins or open web joists. Transverse steel frames are composed of steel beams or trusses and columns. Longitudinal frames may have tension-only diagonal bracing to resist lateral loads. Exterior walls are usually light metal siding or transite panels.

6.3.2 Performance Characteristics

These buildings are typically quite flexible because of their light framing. This flexibility can result in large drifts and lead to nonstructural damage. In general, however, these buildings have an excellent performance record and the lowest expected damage ratio of any building. The following statements discuss some specific performance characteristics that these buildings may exhibit:

1. Insufficient capacity of the longitudinal tension-only braces can lead to their stretching or rupture, especially at the connections. This results in the lack of a stable lateral force resisting system in the longitudinal direction, although rarely does this condition lead to collapse.

2. Exterior wall panels can fall if their connections to the building frames have insufficient strength, displacement capacity and/or ductility. Panel elements can also fail if they cannot accommodate the interstory drift. Panels with insufficient joint size can work against each other.

3. Glass in large skylights can shatter due to diaphragm deformations.

4. Inadequate connection to the foundation may allow the building columns to slide.

5. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent buildings are not at the same elevation, and where there are no shear walls parallel to the pounding forces located to directly resist these loads.

6. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e. re-entrant corners) if separation joints or special reinforcing has not been provided.

7. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

6.3.3 Examples of Building Performance

Included herein is a short description of the performance exhibited in a past earthquake by a building of this classification. This description includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This is not intended to be all inclusive, but rather to demonstrate past performance and provide a reference for more information on this building type.

1. Light metal industrial buildings (general), Coalinga, California 1983 (CDMG, 1983). When clad with sheathing, these structures generally performed well. Some rod braces may have broken or buckled. Weight of heavy transite panels in some cases caused substantial swaying and loss of panels on longitudinal walls. MM VIII.

6.3.4 Loads and Load Paths

Gravity loads are transferred from the roofing elements to steel purlins or open web joists that span between main framing lines. The main transverse beams or trusses then transfer loads to the steel columns on the building perimeter and/or interior.

Lateral loads are transferred from the decking into the transverse frames that resist the forces through moment frame action. Longitudinal lateral forces are resisted by truss action of the perimeter beams and columns and the diagonal bracing.

Typical roof dead loads may range between 5 to 10 psf. Roof live loads are typically taken as 20 psf before allowable reductions for large supported areas.

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 0.4 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.

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.

6.4 Seismic Evaluation of Steel Frame Buildings with Cast-in-Place Concrete Walls

6.4.1 Building Description

This building includes floor and roof diaphragms that are generally composed of either metal decking with concrete fill or cast-in-place concrete slabs. Floor framing consists of steel beams or open web joists and girders. Steel columns combine with the girders to form a gravity load resisting frame. Cast-in-place concrete shear walls, which may or may not be bearing walls, form the primary lateral load resisting system. Exterior walls may be either metal, concrete, or precast concrete panels.

6.4.2 Performance Characteristics

These buildings are typically stiffer than either moment resisting or braced steel frame structures. This increased stiffness results in less interstory drift and subsequent damage to nonstructural elements. The following statements list some specific performance characteristics that these buildings may exhibit:

1. Large seismic events can cause shear cracks and distress around openings. Spalling of concrete is possible if a sufficient number of high amplitude cycles occur.

2. Walls that are discontinuous may lead to column distress.

3. Wall construction joints can create planes of horizontal weakness that may lead to shear failure at a force level well below the expected capacity.

4. Insufficient chord steel lap lengths can lead to wall bending failures.

5. Insufficient confining steel at chord lap locations may cause splices to fail before full development of the bars is attained.

6. Exterior precast wall panels can fall if their connection to the building frames have insufficient strength, displacement capacity, and/or ductility. Panel elements can also fall if they cannot accommodate the interstory drift. Panels with insufficient joint size can work against each other.

7. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent buildings are not at the same elevation, and where there are no shear walls parallel to the pounding forces located to directly resist these loads.

8. Concrete parapets that are unreinforced and have large height to thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

9. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

10. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

6.4.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity, if available, in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Providence Hospital, Anchorage 1964 (C&GS, 1967, p. 71). Five-story simple steel framing, complete, except west concrete shear wall, which also acts as a bearing wall. Metal deck and fill on floors. Designed for Zone 3 of 1958 UBC. Damage ratio of 2-1/2 percent was mostly due to distress around duct openings in shear walls.

2. Alaska Psychiatric Institute, Anchorage 1964 (C&GS, 1967, p. 125). Five-story simple steel frame was essentially complete. Metal deck and concrete slabs. Concrete shear walls exhibited minor cracking and some pounding. Designed for Zone 3 of 1958 UBC.

3. Anchorage Westward Hotel, Anchorage 1964 (C&GS, 1967, p. 39). Several units, tower fourteen stories, simple steel frame but only erection columns in bearing shear walls, concrete slab. 12-1/2 percent damage ratio. No collapse or casualties. Damage to shear walls, and stairs, and some pounding. Designed for Zone 3 of 1958 UBC.

4. Hill Building, Anchorage 1964 (C&GS, 1967, p. 32). Eight-story office building, simple steel framing around CIP concrete core, CIP concrete slabs. Failure of poor quality core concrete caused several inches of settlement. Designed for Zone 3 of UBC (completed in 1962).

5. Cordova Building, Anchorage 1964 (C&GS, 1967, p. 59). Six-story steel moment frame for transverse forces, cast-in-place concrete bearing wall core, with metal curtain walls except 4 inches cast-in-place concrete curtain walls at ends. Steel joists supported metal decking with 2-1/2 inches fill. Four-inch curtain walls cracked and one steel column distorted badly. Moment connections at building front had local buckling and core walls were damaged at base.

6. Four Seasons Apartment, Anchorage 1964 (C&GS, 1967, p. 176). Six-story building, two concrete cores, with steel columns, and post-tensioned concrete lift slabs. Cores overturned due to brittle tension lap at base. Designed for Zone 3 of UBC. Collapsed.

7. Alaska Methodist University, Anchorage 1964 (C&GS, 1967, p. 168). Two buildings. One is a three-story structure with steel pipe columns and beams using composite construction concrete floor slabs and cast-in-place concrete shear walls. Roof has metal decking on open web steel joists. The other building is a three story structure with reinforced concrete slabs supported on concrete or reinforced concrete block bearing and shear walls. One side has precast concrete structural wall panel. Well tied. Designed for Zone 3, 1958 UBC. Very little damage, 3 percent damage ratio.

8. Coalinga High School, Boys' Gym, Coalinga, California 1983 (CDMG, 1983, pp. 42-46). Steel rigid frame, 2 to 8 feet wide reinforced concrete piers at each side wall. Composition roofing supported at 3 feet 8 inches by poured-in-place gypsum. Eight-foot wall piers had diagonal cracks away from windows at lower corners. MM VIII.

6.4.4 Loads and Load Paths

Gravity loads are transferred from the floor slabs to the floor framing elements composed of steel beams or open web joists. Floor girders are supported by steel columns that transfer loads to the foundation. Concrete shear walls may support tributary gravity loads, or may have steel framing elements embedded for gravity loads.

Lateral loads are transferred from the floor diaphragm to collector elements and to the shear walls. The shear walls then transfer loads to the foundation elements. Coupling beams between wall elements may provide combined wall action. The steel gravity frame may provide a secondary lateral force resisting system, depending on the completeness of the frame and the moment capacity of the beam-column connections.

Typical floor dead weights may range between 70 and 120 psf. Live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy.

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 Section 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 0.4 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 ACI 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

6.5 Seismic Evaluation of Steel Frame Buildings with Infilled Walls of Unreinforced Masonry

6.5.1 Building Description

This building type includes floor and roof diaphragms that may be composed of straight or diagonally sheathed wood supported by wood subframing in older construction. More recent construction could consist of plywood diaphragms. Cast-in-place concrete slabs and metal deck and concrete fill may also be used. The gravity load bearing system consists of a complete steel frame. The exterior walls and possibly some interior partitions are composed of unreinforced masonry that has been infilled into the steel frames. This infill may consist of solid clay brick, concrete block, or hollow clay tile masonry. In some instances, to provide natural lighting, the masonry on exterior lines does not extend to the soffit of the floor beams. The exterior infill may include an unbonded veneer course. The infilled walls may not continue to the base of the building. In many cases of early construction, the exterior wythes may be joined to the interior wythes only by the mortar placed in the collar joint. In other cases, different wythes may be tied together by using bricks laid with the long dimension across the collar joint (headers). Recent practice often leaves the collar joint free of mortar (cavity construction) with the bonding between wythes dependent on light gage metal ties. Anchorage of the infilled walls to the steel frames may consist of light metal ties or solely the bond provided by the mortar at the interface. Infilled walls may change the initial response of the frame structure to that of a shear wall building if they are not sufficiently isolated from the frames.

6.5.2 Performance Characteristics

Steel frame buildings with infilled walls of unreinforced masonry have performed well when the infilled walls were well anchored into the steel frames. The infilled walls alter the structural response because their stiffness concentrates the lateral resistance in the frame lines that have been infilled. The infilled panels, especially those of hollow clay tile, have often been severely damaged during intense shaking. The following statements discuss some specific performance characteristics that these buildings may exhibit:

1. Unreinforced masonry appendages or cornices can fall.

2. Infilled walls that are insufficiently anchored to the diaphragms or steel frames can separate from the building and fall. Unanchored gable ends of masonry walls are especially susceptible to this problem.

3. Exterior veneer courses can separate from the masonry wall and fall.

4. Infilled walls adequately connected to the diaphragms and/or frames can exhibit large diagonal cracks in the infill due to in-plane forces.

5. Infilled walls that are not continuous to the base of the building can cause a serious vertical strength discontinuity. This discontinuity can lead to concentration of inelastic action in the frame elements where the infill is discontinued.

6. Infilled walls can exhibit large diagonal cracks in the masonry due to in-plane forces. With continued cycling, spalling of the infill may occur. Hollow clay tile partitions are especially susceptible to this type of damage.

7. Unreinforced masonry parapets that have large height-to-thickness ratios, or are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

8. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

6.5.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. AETNA Building, San Francisco 1906 (USGS, 1907, pp. 31-32, 76-77). Five-story structure, founded on piles, with granite bearing walls. Also pressed brick and terra-cotta bearing walls. Steel columns and girders. Reinforced concrete floors. Relatively little damage suffered. Cracking of brick walls. One panel of fifth floor collapsed due primarily to heat of fire. MM IX.

2. Bullock and Jones Building, San Francisco 1906 (USGS 1907, pp. 33, 80-81). Eight-story steel frame. Reinforced concrete floors, hollow tile partitions, terra-cotta and brick bearing walls. Reinforced concrete floor arches haunched between steel girders but not continuous over girders. Considerable exterior damage. MM VII.

3. City Hall, San Francisco 1906 (USGS, 1907, pp. 35-36, 84-89). Brick with steel floor beams, corrugated iron arches and cinder concrete filling. Cast-iron columns. Heavy brick interior partitions. Reported that the masonry had been reinforced with embedded steel bars but this was unverified. Projecting pilasters on exterior walls badly cracked. Earthquake tended to shear off these pilasters even though they were built of same material as wall and well bonded to it. This could be caused by wall rocking onto pilaster and shearing it from the base up. Many diagonal braces in tower were stretched beyond their elastic limit. Brick work was shaken from central tower exposing steel from beneath. Quality of mortar was questionable. MM VIII.

4. Jackson Brewing Company Building, San Francisco 1906 (USGS, 1907, p. 39). Under construction. Brick walls with lime mortar were destroyed. Steel beams on cast-iron columns. Insufficient steel member connections, girders and beams resting on walls without any ties. Eastern half of concrete floor slabs (six inches thick) were unreinforced. Tower and building collapsed. MM VIII-IX.

5. Hall of Justice, San Francisco 1906 (USGS, 1907, pp. 39, 93). Steel frame and reinforced concrete cinder floors. Building largely destroyed by earthquake. The cupola supported by light steel angles collapsed from the heat of fire after being racked by earthquake. Brick walls laid on lime mortar and floor panels stiffened by 5 inch-by-1/2 inch steel bands. Suspended ceilings of plastered expanded metal lath. Partitions of 3-inch expanded metal plastered. Suspended ceilings failed. MM VIII.

6. Kamm Building, San Francisco 1906 (USGS, 1907, pp. 39-40, 93). Seven-story L-shaped plan. Steel frame, reinforced concrete floors, hollow partitions, sandstone bearing walls, suspended ceilings. MM VIII.

7. Post Office Building, San Francisco 1906 (USGS, 1907, pp. 44-45, 97-103). Three stories on foundation of steel beams encased in concrete. Each column had its own footing, some extending as far as 30 feet deep. Steel frame, reinforced concrete floors, suspended ceilings. Partitions and interior walls of hollow terra-cotta tile. Well-anchored exterior granite walls. This building sustained some of the most severe shaking in all of the city, but suffered relatively little earthquake damage. Ground settled 5 feet at one corner - building slightly cracked at this point but only settled 1-3/4 inches. Stones shaken loose from exterior walls, cracking. Some chimneys thrown down. Interior walls of hollow tile, although strengthened by plaster finish, were cracked extensively. MM IX.

8. Hillside Apartments, Anchorage 1964 (C&GS, 1967, p. 130). Pipe column and simple steel beam frame. Unreinforced, unanchored concrete block walls and partitions, concrete slab floor, steel joist roof, split level. Not designed for earthquake. Excessive damage required the building to be razed.

9. KOHL Building, San Francisco 1906 (USGS, 1907, p.40) An 11-story steel frame with concrete floors, hollow tile walls, and store facade. Damage was limited to broken glass, loosening of marble wainscoting and cracks in the facade. MMI VII.

10. James Flood Building, San Francisco 1906 (USGS, 1907 pp 37-92). Twelve-story steel frame with segmental hollow tile floor arches. Terra-cotta partitions and column covering. Some lower columns were found to be slightly buckled and the hollow tile partitions were cracked. Additional damage includes cracking of the sandstone piers at several entrances. MMI VII.

11. Aronson Building, San Francisco 1906 (USGS, 1907, pp. 32-78). Ten-story steel framed structure with reinforced cinder-concrete floors. Terra-cotta partitions and column covering. Most damage was caused by the fire. Earthquake caused minimal damage limited to wall cracking. MMI VII.

6.5.4 Loads and Load Paths

Gravity loads are transferred from the floor and roof diaphragms to the subframing which is supported by the steel frames. The steel frames may also support the weight of the infilled masonry walls and/or partitions.

Past earthquakes have shown that infilled masonry walls and partitions drastically alter the seismic response of this building type. In the elastic range, the stiffness of the infill causes the building to respond as a shear wall structure. Once cracks form along the boundary between the infill and the frame, the response is similar to that of a braced frame with the infill in compression acting as the diagonal elements. If the cyclic response continues, the masonry cracks become more severe, and spalling may commence. As the stiffness of the masonry infill degrades, the steel frames may begin to resist the lateral loads through frame action. Note that this scenario of response is often not that which was anticipated by the original designer. In many cases, the stiffness of the infilled walls was ignored and only the frame action of the steel elements was considered.

Typical floor dead weights depend on the diaphragm material. Wood floors may weigh between 15 and 40 psf; concrete floors may weigh between 90 and 130 psf. Roof live loads are typically taken as 20 psf, and floor live loads may range from 40 to 100 psf, depending on the occupancy. These live loads may be reduced for members that support large areas. Typical brick masonry weighs approximately 120 pcf.

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 Section 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 0.4 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. They were not designed for in-plane loadings which could produce distress and create a potential falling hazard if the diaphragm stresses are large. 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.

Evaluate the diaphragm shear forces to be resisted by the clay tile arch floors. If they exceed 120 pounds per foot, then perform further investigations of the materials.

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. Check for spalled joints, tie rod size, spacing and condition, steel beam condition, floor cracks and loose soffit tiles. For more information of this form of construction, see Kipper (1906, 1900-1920) and Sweets (1906).

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".

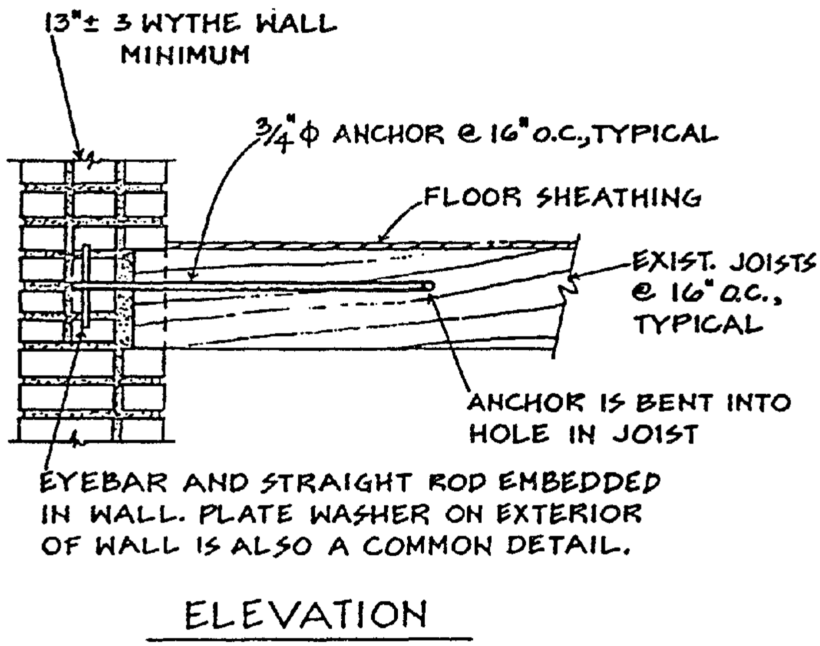
Recommended C/D Ratio: 1.0.

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.



Typical Detail for "Government Anchors". These Anchors May Also Be Known as "Dog Anchors".

FIGURE 6.1

SECTION 7

SEISMIC EVALUATION OF CAST-IN-PLACE CONCRETE BUILDINGS

7.1 Seismic Evaluation of Moment Resisting Cast-in-Place Concrete Buildings

7.1.1 Building Description

This building type includes floor and roof diaphragms that are typically composed of cast-in-place concrete slabs. The slabs are generally supported by a system of beams, one-way joists, two-way waffle joists, or flat slabs. Major floor framing elements may not have continuous top and bottom reinforcing and may or may not be considered to be "ductile concrete." These elements may be post-tensioned. Concrete columns may have large (greater than D/2) tie spacing. The concrete frames provide the primary lateral force resisting system.

7.1.2 Performance Characteristics

Concrete moment resisting frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large interstory drifts that may lead to extensive nonstructural damage. If the concrete columns have a shear capacity below the moment capacity, brittle column failure can occur, possibly resulting in collapse. The following statements list some specific performance characteristics that these buildings may exhibit:

1. Large tie spacing in columns can lead to a lack of confinement for the concrete core and/or shear failures.

2. Insufficient column lap lengths can cause concrete to spall.

3. Location of inadequate splices for all column bars at the same section can lead to column failure.

4. If the column shear strength is insufficient to develop the full moment hinge capacity, the column can exhibit a brittle shear failure.

5. Insufficient anchorage of shear tie reinforcing in column cores can prevent the column from developing its full shear capacity.

6. Lack of continuous beam reinforcement can cause hinge formation during load reversals.

7. Inadequate reinforcing of beam-column joints or location of beam bar splices at columns can lead to joint failures.

8. Foundation dowels that are insufficient to develop the capacity of the column steel above can icad to local column distress.

9. Offsets or eccentricities between girders and columns of exterior frames can cause unanticipated forces such as panel zone torsion that may lead to distress in these frames.

10. Use of bent-up longitudinal reinforcing in beams as shear reinforcement can result in shear failure during load reversal.

11. The relatively low stiffness of the frames can lead to excessive interstory drifts. These large drifts can cause damage to nonstructural items such as partitions, ceilings, lights, windows, etc.

12. Exterior precast wall panels can fall if their connection to the building frames have insufficient strength, displacement capacity, and/or ductility. Panel elements can also fail if they cannot accommodate the interstory drift. Panels with insufficient joint size can work against each other.

13. If glazing is not sufficiently isolated from structural actions, it can fail and fall out onto streets sidewalks, or the adjacent property.

14. Insufficient isolation of nonstructural masonry walls from the structural system may alter the assumed structural response. If these infilled walls are not properly detailed, they may fail due to the lateral forces. See Section 7.3.

15. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent buildings are not at the same elevation, and where there are no shear walls that are parallel to the pounding forces located to directly resist these loads.

16. Concrete parapets that are unreinforced and have large height-to-thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

17. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

18. Buildings with abrupt changes in lateral resistance have often performed poorly in past earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

7.1.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity, if available in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Sheraton Universal Hotel, San Fernando 1971 (Murphy, 1973, p. 307), Twenty-story structure, about 58 feet x 184 feet, with cast-in-place concrete girders and columns detailed as ductile under 1966 L.A. City Code (similar to 1970 UBC). Nineteen miles from fault, this building suffered minor damage ($2,000). Peak base acceleration was 18 percent g. MM VII.

2. Bank of California, San Fernando 1971 (Murphy, 1973, p. 327). Twelve-story cast-in-place ductile concrete moment frame built to 1969 L.A. Building Code. This building, which was 17 miles from fault, experienced a ground floor acceleration of 23 percent g. Damaged members included columns, girder stubs, cracking in spandrels and floor slab. This $4,000,000 building sustained $40,000 in damage. MM VII.

3. Banco Central, Managua 1972 (Wyllie et al, 1974, p. 1077). Fifteen-story, non-ductile cast-in-place concrete moment frame designed for 12 percent g. Columns cracked, diaphragms tore, and partial collapse occurred. Much nonstructural damage occurred. Did not collapse.

4. Telcor Building, Managua 1972 (Wyllie et al, 1974, p. 1079). Seven-story concrete non-ductile cast-in-place concrete frame, with precast beams, topping slab, and some post-tensioning in girders. Designed to resist earthquakes. Major damage occurred, requiring the building to be torn down.

5. Airport Control Tower, Anchorage 1964 (C&GS, 1967, p. 96). Five-story cast-in-place non-ductile moment resisting frames with concrete floor slabs and light metal panel walls. Collapsed, with one person killed.

6. Social Services Building, Santa Rosa 1969 (Steinbrugge et al, 1970, p. 25). Two-story, cast-in-place concrete non-ductile moment resisting frame with extra ties; can be classified as an intermediate (old California style) waffle slab. Designed for Zone 3 of 1964 UBC. Exterior nonstructural metal panels, 18-inch x 18-inch tied columns cracked seriously, stair damaged. MM VII-VIII.

7. Olive View Hospital, San Fernando 1971 (Murphy, 1973, p. 255). Four stories of shear wall structure supported by two levels of non-ductile concrete frames. Spiral columns held up main building with 24-inch offset. Four stair towers (separate from building) on tied columns collapsed. Many types of failure. Building razed after earthquake. MM VIII-XI.

8. Olive View Psychiatric Unit, San Fernando 1971 (Murphy, 1973, p. 290). Two-story non-ductile frame, concrete with tied columns. Building collapsed. MM VII-XI.

9. Olive View Ambulance Canopy, San Fernando 1971 (Murphy, 1973, p. 289). One-story, non-ductile frame. Collapsed. MM VIII-XI.

10. Holiday Inn (Orion), San Fernando 1971 (Murphy, 1973, p. 359). Seven-story cast-in-place concrete. Cast-in-place 8-1/2 inch concrete slab (20 feet) on columns with exterior spandrel beams. Interior gypsum partitions and plaster exteriors. All frames act to resist lateral forces. Thirteen miles from fault, this building underwent a ground acceleration of 25 percent g. Designed in 1965 at a cost of $1.3 million; 1971 damage was $145,000, mostly nonstructural.

11. Holiday Inn (Marengo), San Fernando 1971 (Murphy, 1973, p. 395). Construction nearly identical with Orion Holiday Inn (above). Twenty-six miles from fault, this building underwent 15 percent g peak ground floor acceleration, $95,000 to repair.

12. Muir Medical Center, San Fernando 1971 (Murphy, 1973, p. 481). Eleven-story cast-in-place concrete building with flat slabs and a perimeter moment frame, built in 1968, 200,000 square feet, $4.5 million. Design code similar to 1964 UBC. Designed as space frame with K = 0.67. Although it did not meet ductile requirements, the building had many added features (ties, some spiral columns, anchors, etc.) that helped to provide ductile performance. Twenty-one miles south of epicenter, this building underwent a peak ground acceleration of 10 percent g. No structural damage, but non-structural repairs cost $2,000. MM VII.

13. Union Bank Building, San Fernando 1971 (Murphy, 1973, p. 629). Thirteen-story cast-in-place slab, beams and tied columns, non-ductile moment-resisting frame. Two stories below grade. Designed in 1964 for L.A. City Code. Four corner columns were damaged at second floor (23 feet 8 inches story height). Second floor spandrels also cracked. Non-structural damage to partitions, ceilings, floor tile, and veneer. Total repairs cost approximately $100,000. Seventeen miles from fault, across the street from Bank of California, where measured peak ground acceleration was 23 percent g. MM VII.

14. Patrick Henry Junior High, San Fernando 1971 (Murphy, 1973, p. 679). Two-story concrete arcade between classroom wings. Columns were damaged, but did not collapse. Shored and torn down. MM VIII-XI.

7.1.4 Loads and Load Paths

Gravity loads are transferred from the floor slabs to floor framing elements such as one-way joists or waffle joists, or through flat slab action to large beams or girders. Concrete columns support the major floor framing elements and transfer the gravity loads to the foundation.

Lateral loads are transferred from the floor diaphragms to the moment resisting frames. These frames may or may not be ductile depending on their configuration and joint details. Column shear strengths that are not sufficient to develop the joint moment capacities of the beams at the joint can lead to undesirable shear failures.

Typical floor dead weights may range from 90 to 130 psf. Live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy.

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:

$$V_{AVG} = \left(\frac{n_c}{n_c - n_f}\right) \bullet \left(\frac{V_j}{A_c}\right)$$

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 Section 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 are 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:

$$\Delta = \left(\frac{k_b + k_c}{k_b k_c}\right)\left(\frac{h}{4500}\right) V_c$$

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.

7.1.5.3 Evaluation of Materials

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 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 7.1.5.2: 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 considerations 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 0.4 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 away 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. Per 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 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 7.1.5.22: 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 recommended. 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 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.

7.2 Seismic Evaluation of Cast-in-Place Concrete Shear Wall Buildings

7.2.1 Building Description

This building type contains floor and roof diaphragms typically composed of cast-in-place concrete slabs. The slabs are generally supported by a system of beams, one-way joists, two-way waffle joists, or flat slabs. Concrete columns or bearing walls support the majority of the gravity loads. Cast-in-place concrete shear walls, which may be bearing walls, provide the primary lateral force resisting system. The response of these walls depends on their location, configuration, the number and size of openings, and reinforcement details.

7.2.2 Performance Characteristics

Concrete shear wall buildings are typically stiffer than buildings that resist lateral forces through moment frame action. This increased stiffness results in less interstory drift and subsequent damage to nonstructural elements. The following statements discuss some specific performance characteristics that these building may exhibit:

1. Large seismic events can cause shear cracks and distress around openings. Spalling of concrete is possible if a sufficient number of large cycles occur.

2. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories. Shear walls that are not continuous to the base of the building may lead to column shear failures.

3. Wall construction joints can create planes of horizontal weakness that may lead to shear failure at a force level well below the expected capacity.

4. Insufficient chord steel and/or lap lengths can lead to wall bending failures.

5. Insufficient confining steel at chord lap locations may cause splices to fall before full development of the bars is attained.

6. Insufficient vertical stirrups in coupling beams results in a lack of shear resistance that can cause a loss of coupling action. Portions of the coupling beam can spall off and fall onto the area below.

7. Exterior precast wall panels can fall if their connections to the building frames have insufficient strength, displacement capacity, and/or ductility. Panel elements can also fail if they cannot accommodate the interstory drift. Panels with insufficient joint size can work against each other.

8. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent buildings are not at the same elevation, and where there are no shear walls parallel to the pounding forces located to directly resist these loads.

9. Concrete parapets that are unreinforced and have large height-to-thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

10. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

7.2.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity, if available, in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. St. Mary's Residence (old Providence Hospital), Anchorage 1964 (C&GS, 1967, pp. 17 and 180). Three-story cast-in-place concrete with cast-in-place concrete walls, built in 1939. No damage.

2. Alaska Native Hospital, Anchorage 1964 (C&GS, 1967, p. 18). Five-story all cast-in-place concrete, including walls. No damage.

3. Knik Arms Apartment, Anchorage 1964 (C&GS, 1967, p. 157). Six-story all cast-in-place concrete, including walls. No damage.

4. Mt. McKinley and 1200 L Apartments, Anchorage 1964 (C&GS, 1967, p. 76). Cast-in-place concrete core, walls and floors. Fourteen stories, designed for Zone 2 of 1952 UBC. Much spandrel, column and stair tower cracking, damaged between 30 and 40 percent of replacement cost. Did not collapse and no casualties.

5. J.C. Penney, Anchorage 1964 (C&GS, 1967, p. 112). Five-story all cast-in-place concrete floors and shear walls. Precast on exterior walls with very eccentric shear wall layout, and improper construction of shear walls. Partial collapse occurred, resulting in four deaths. Building was later demolished.

6. Hodge Building, Whittier, Alaska 1964 (C&GS, 1967, p. 192). Fourteen-story building with cast-in-place concrete walls, floors, and slabs. Coupling beams in shear walls were cracked, but the building basically exhibited good performance.

7. Museum for Antique Cars, San Fernando 1971 (Murphy, 1973, p. 129). Five-story building with cast-in-place concrete walls, slabs, beams, and columns, under construction at the time of the earthquake. Few openings in walls. Floor construction and lightweight concrete walls cracked, construction joints where lightweight slabs joined walls moved. Top had permanent displacement. Damage ratio of 15 percent. MM VIII-XI.

8. Pacoima Memorial Lutheran Hospital, San Fernando 1971 (Murphy, 1973, p. 205). Five-level building with cast-in-place concrete floors, columns, and shear walls. Structure is complicated in plan. Designed for earthquake and future expansion. Three units were separated by expansion joints. Major damage to all units, including shear in coupling beams. No collapse, but partially demolished rather than repaired. MM VIII-XI.

9. Veteran's Administration Hospital, Sylmar, San Fernando 1971 (Murphy, 1973, p. 655). There were several concrete slab and shear wall buildings, built in 1930 and later. No collapse. MM VIII-XI.

10. Banco de America, Managua 1972 (Wyllie et al, 1974, p. 1073). Seventeen-story cast-in-place concrete shear wall structure. Designed for 1967 SEAOC Blue Book, Zone 3. Failure of coupling beams, cracks in shear walls.

11. Indian Hills Medical Center, San Fernando 1971 (Murphy, 1973, p. 219). Seven-story, "complete" cast-in-place non-ductile concrete frame with concrete shear walls. Designed for 1966 L.A. Building Code, K = 1.0. Damage to shear walls, slabs, and frame. Damage ratio was 9 percent. MM VIII-XI.

12. Holy Cross Hospital, San Fernando 1911 (Murphy, 1973, p. 235). Three buildings. Main building is a seven-story, cast-in-place concrete structure (joists, girders, and shear walls). Shear walls are bearing (no columns), designed for 1959 L.A. Building Code. Some shear walls are discontinuous at second floor. Diaphragms cracked badly due to inadequate collectors. Shear walls also cracked and displaced, but no collapse occurred, Major damage, required evacuation. Damage ratio was 48 percent. MM VIII-XI.

13. Certified Life Building, San Fernando 1971 (Murphy, 1973, p. 541). Fourteen-story, cast-in-place office tower with shear walls, designed in 1966, $3 million construction cost, 170,000 square feet, 17 miles south of epicenter. Foundations included battered piles. Floors were interior flat slabs supported by columns. Designed in accordance with 1964 L.A. Building Code, K = 1.33 (used J factor). No structural damage, except hairline cracks in shear walls. Gypsum partitions cracked 1/8-inch to 3/16-inch. Some mechanical equipment was damaged. Peak ground acceleration of 26 percent g. MM VII.

14. Dawson Elementary School Coalinga, California 1983 (CDMG, 1983, pp. 40-41). Administration building and classroom wings had wood truss rafters supporting diagonal sheathing and wood framing. Exterior concrete walls. Minor plaster cracking and broken windows. Learning Center had rod trusses supporting diaphragm; it sustained no significant damage. MM VIII.

15. Sunset Elementary School, Coalinga, California 1983 (CDMG, 1983, p. 41). Classroom wings, administration office, and auditorium had wood roofing and reinforced concrete walls. Recreation building had rod braced steel rigid frames and concrete walls. Cafeteria had steel roof trusses and steel beams supporting wood joists and diagonal sheathing with reinforced concrete walls. In all cases, no structural damage. Minor architectural damage, including broken windows. MM VIII.

16. West Hills College, Coalinga, California 1983 (CDMG, 1983, pp. 46-47). Speech Arts Building had plywood roof diaphragm on steel trusses supported by reinforced concrete columns or walls. Plaster ceiling cracked. Concrete spalling at pilasters supporting steel trusses. Supplementary column ties around truss anchor bolts were omitted. MM VIII.

17. Coalinga High School, Coalinga, California 1983 (CDMG, 1983, pp. 42-46). Classroom and administration complex had exterior reinforced concrete walls, wood framed roof or wood joists bearing on wood stud walls as well as concrete shear walls. East classroom wing had exterior reinforced concrete walls. First floor had wood framing with reinforced concrete corridor. Second floor had a reinforced concrete diaphragm on reinforced concrete walls. Wood roof. Roofing tiles displaced. Spilled chemicals caused interior damage. The tranverse direction of the Girls' Gym (shower and locker building) had steel rigid frames with reinforced concrete end walls. In the longitudinal direction there were reinforced concrete walls with a steel rod braced diaphragm. Tie rods failed in end bays. Shop Building had wood roof joists with steel rod-braced diaphragms bearing on reinforced concrete columns and walls. Rod/clevis connection was inadequate in one roof bay. Exterior plaster cracked. Interior plaster partitions and wall tile cracked. MM VIII.

18. Snaidero Office Building, Friuli, Italy 1976 (Stratta and Wyllie, 1979, pp. 20-23). Six-story structure. Central reinforced concrete core, and four large interior columns support heavy roof beams that support tension hangers at perimeter of upper four floors. Concrete core walls heavily damaged. Floor slab cracked between columns and closely located core walls as floor system tried to transfer overturning forces. MM VIII-IX.

19. Lawrence Livermore Laboratory, Livermore, California 1980, 8 miles from epicenter (Degenkolb Associates, 1980, pp. 15-17). Building 113: Five-story building (90-foot by 90-foot floor plan), two-way waffle slab with columns at 30 feet centers, reinforced concrete core walls. Diagonal cracks in core walls. Horizontal cracks at construction joints within stairwells. Cracks in fourth and fifth floors near corners on concrete core near perimeter frame occurred as slabs attempted to transfer overturning forces from walls to perimeter frames. Non-structural cracking and damage also occurred. Building 332: One-story building with 10-inch exterior reinforced concrete walls. Interior has 9-inch reinforced concrete walls. Several hairline diagonal cracks were noted after the earthquake. Some movement and damage to ceiling tile and floor plate noted at expansion joint between older and newer buildings.

7.2.4 Loads and Load Paths

Gravity loads are transferred from the floor slabs to the floor framing elements such as one-way joists or waffle joists, or through flat slab action to larger beams, walls, or columns. Concrete columns and walls support the major floor framing elements and transfer the gravity loads to the foundation.

Lateral loads are transferred from the floor diaphragms to the shear walls, possibly through collector elements. The walls transmit the loads through shear and bending to foundation elements. Walls with shear capacities that are not sufficient to develop the bending capacities may lead to non-ductile inelastic response.

Typical floor dead weights may range from 90 to 130 psf. Live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy.

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

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 Section 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. (See Figure 7.1 for Detail of Coil Loop Anchor.)

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 posttensioning 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 0.4 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.

Recommended 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.

7.3 Seismic Evaluation of Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry

7.3.1 Building Description

This building type includes floor and roof diaphragms that are typically composed of cast-in-place reinforced concrete. The gravity load bearing system consists of a complete concrete frame that may be a flat plate and column system. The frame elements are typically not provided with the reinforcing details necessary to be considered ductile. The exterior walls and possibly some interior partitions are composed of unreinforced masonry that has been infilled into the concrete frames. This infill may consist of solid clay brick, concrete block, or hollow clay tile masonry. In some instances, to provide natural lighting, the masonry on exterior lines does not extend to the soffit of the floor beams. The exterior infill may include an unbonded veneer course. The infilled walls may not continue to the base of the building. In many cases of early construction, the exterior wythes may be joined to the interior wythes only by the mortar placed in the collar joint. In other cases, different wythes may be tied together by using bricks laid with the long dimension across the collar joint (leaders). Recent practice often leaves the collar joint free of mortar (cavity construction) with the bonding between wythes dependent on light gage metal ties, Anchorage of the infilled walls to the concrete frames may include light metal ties or solely the bond provided by the mortar at the interface. Infilled walls may change the initial response of the structure to that of a shear wall building if they are not sufficiently isolated from the frames.

7.3.2 Performance Characteristics

Concrete frame buildings with infilled walls of unreinforced masonry may not provide adequate performance during high intensity earthquakes, The infilled walls alter the structural response because their stiffness concentrates the lateral resistance in the frame lines that have been infilled. This concentration can lead to local distress in the frame elements. Frames that consist of flat slab and column elements have been especially susceptible to damage because of their flexibility. The infilled panels, especially those of hollow clay tile, have often been severely damaged during intense shaking. The following statements discuss some specific performance characteristics that these buildings may exhibit:

1. Unreinforced masonry appendages or cornices can fall.

2. Infilled walls that are insufficiently anchored to the diaphragms or frames can separate from the building and fall. Unanchored gable ends of masonry walls are especially susceptible to this problem.

3. Exterior veneer courses can separate from the masonry wall and fall.

4. Columns can fail in shear due to the lack of ties that may result in collapse.

5. Infilled walls that do not extend to the soffit of the beam above can cause short column shear failures.

6. Infilled walls that are not continuous to the base of the building can cause a vertical strength discontinuity. This discontinuity can lead to concentration of inelastic action in the frame elements where the infill is discontinued. Column shear failures may also result.

7. Infilled walls can exhibit large diagonal cracks in the masonry due to in-plane forces. With continued cycling, spalling of the infill may occur. Hollow clay tile partitions are especially susceptible to this type of damage.

8. After cracking occurs along the boundary between the infill and the frame, the masonry acts like a compression diagonal in a braced frame. This diagonal strut action may lead to local force concentrations at the frame joints, which can cause column shear failures in some cases.

9. Unreinforced masonry parapets that have large height-to-thickness ratios, or are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

10. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

7.3.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Elite Condominiums, Friuli Italy 1976 (Stratta and Wyllie, 1979, pp. 14-16). Six-story, reinforced concrete frame structure with infilled hollow tile walls. Considerable distress at first story columns. Clay tile infilled walls were fewer at first story, giving first story less strength and stiffness than upper floors. Clay tile walls were badly damaged. MM VIII-IX.

2. Gemona Hospital, Friuli, Italy 1976 (Stratta and Wyllie, 1979, pp. 24-29). Ten story building divided into numerous small structures by expansion joints. Reinforced concrete frame with some reinforced concrete shear walls. Hollow clay tile infilled walls. Considerable spalling of tile finish on exterior walls. Damage to interior and exterior clay tile walls. Damage due to pounding at expansion joints. Considerable leaning in second story of five-story wing due to far less clay tile walls than in other stories, creating abrupt decrease in stiffness. Distress in reinforced concrete shaft at one edge of floor of five-story wing. This shaft caused torsional forces, but added to overall stability. Residual gap of 12 inches at roof expansion joint due to leaning wings. Overturning forces initiated failure at columns due to lack of column ties in joints. MM VIII-IX.

3. Reinforced Concrete Frame Buildings (general), El Asnam, Algeria 1980 (EERI, 1983 pp. 4.1-4.24). Heavy roof and floor systems supported by large, stiff girders and light columns. Stiff infilled masonry walls were stopped at first floor level. Open crawl spaces created short column condition in many cases. Detailing of frame elements was poor. Joint steel, bar anchorage, and splices were all inadequate. Concrete quality was poor. The few shear walls were typically unsymmetrically placed. Insufficient expansion joints allowed for battering of floors. Damage to these buildings was severe. Many collapses due to short column shear failures. MM IX-X.

4. Hotel du Cheliff, El Asnam, Algeria 1980 (EERI, 1983, p. 4.41). Two-story irregular plan. Finished in 1962. Moment resisting, cast-in-place concrete frames with weak columns and strong girders. Heavy masonry walls and partitions, Building destroyed due to failure of columns that were inadequate to resist large shear forces. MM IX-X.

5. Maison de la Culture, El Asnam, Algeria 1980 (EERI, 1983, pp. 4.41-4.43). Four building units, four stories each, 60 percent construction completed. Moment resisting frame of irregular plan and elevation. Long upper-story cantilever with deep reinforced concrete spandrel facade and heavy ornamentation. Columns in each story had different stiffnesses. External columns shorter than the others, weaker than the beams they supported. Shearing of short columns at facade, complete building collapse resulted from high column shear demands combined with poor concrete and inadequate transverse reinforcement. MM IX-X.

6. Sandia Laboratories, Livermore, California 1980 (Degenkolb Associates, 1980, p. 19). Acceleration 0.2 to 0.3 at site. Non-ductile concrete frames with transverse concrete shear walls and irregularly placed concrete block infilled walls. 1955 UBC design. Exterior perimeter frames with 5 feet 6 inch deep beams, concrete block infill and "short" columns. Structural damage included hairline shear cracks in short columns, cracks due to frame action and diagonal and separation cracks in infilled concrete block walls. MM VI.

7. Bekins Van and Storage Company Building, San Francisco I906 (USGS, 1907, p. 33). Reinforced concrete frame. Reinforced concrete floors and brick walls. Six stories. Walls badly cracked but reinforced concrete not damaged by earthquake. MM VIII.

8. "Braced Masonry" construction, San Juan, Argentina 1977 (Poland, in preparation, pp. 5-16). Brick masonry panels used as primary shear transfer, with reinforced concrete beams and columns serving as boundary members, drag chord, and foundation members. Cast-in-place concrete connectors between masonry panels to floor and roof diaphragms. Because of this special construction technique, the general performance of these structures was excellent. Hairline cracks in concrete boundary members and some masonry panels of Cristo Rey Church.

9. Bank of San Juan, San Juan, Argentina 1977 (Poland, in preparation, pp. 36-42). Single-story. Infilled masonry panels around the perimeter. Alternating panels and windows. Two panels at front, four on each side, none in rear. Reinforced concrete two-way roof slab on interior reinforced concrete columns and perimeter panels. Highly eccentric lateral force system in transverse direction. Complete shear failure of braced masonry panels at front, slip-type failures at construction joints on side of structure, and punching through of interior columns. Failures seem to be due to unbalanced system that overstressed front walls. Punching shear failure was a result of additional column bending moments created by roof motion.

10. The Manuel Belgrano School, San Juan, Argentina 1977 (Poland, in preparation, pp. 43-51). "Braced masonry" structure with hollow clay-tile panels. Concrete flat slab roof supported on bearing walls and concrete columns. Severe damage included collapse of portion of roof due to inadequate boundary reinforcement to resist bending; cracking of tile infilled panels in longitudinal direction; failure, of interior and exterior building columns due to inadequate reinforcement; pounding damage at expansion joints; foundation tension failures due to ground spreading caused by liquefaction.

11. Galerie Algerienne, Friuli, Italy 1976 (EERI, 1983, pp. 4.44-4.46). Two-to-four story units separated by thermal expansion joint. The moment resisting space frame was not designed for seismic forces. Interior bays were filled with large hollow brick masonry panels that were not reinforced or anchored to the frame. Some panels failed. One unit collapsed, but the other suffered only severe nonstructural damage. The standing unit contained a properly infilled reinforced concrete frame shaft for the stairwell that was sufficient to resist the lateral forces. The mechanism of failure of collapsed unit seemed to be that the concrete columns could not resist increased shear forces after failure of infilled panels. The columns, which were weaker than girders and poorly reinforced, failed in shear. MM IX-X.

12. Elmendorf Barracks Buildings, Anchorage 1964 (C&GS, 196?, p. 135). Three-story and basement, complete cast-in-place concrete frame, beams, and slabs. Infilled walls of reinforced concrete block. Supposedly designed for earthquake. Walls tried to take load, resulting in damage to block walls and concrete columns. No collapse occurred.

13. Mother Cabrini Giris School, Downtown Los Angeles, San Fernando 1971 (Murphy, 1973, p. 649). Two buildings, one two-story and one three-story. Concrete floors and columns with brick infilled walls and clay tile partitions. Diagonal cracks occurred in walls at windows, and part of the parapet fell. Some outward movement of walls occurred. Loss estimated at $500,000; buildings were demolished.

14. Cotton. Exchange Building, Downtown Los Angeles, San Fernando 1971 (Murphy, 1973, p. 652). Six-story reinforced concrete building built before 1910. Some exterior walls were concrete; others were brick. One wall was a previous brick party wall, later non-bearing. Exterior walls were not anchored. Fifth and sixth floor walls collapsed, lower levels moved outward. Other walls cracked. Walls were replaced, and cracks were repaired. Cost of repairs was $25,000. MM VI.

15. Los Angeles High School, San Fernando 1971 (Murphy, 1973, p. 673). Three- and four-story 1917 school. Concrete floors and frame with unreinforced brick walls. Severe damage. MMVI.

16. California Garage, Long Beach 1933 (Green, 1933, pp. 560-562). Three-story concrete beam and joist structure with brick curtain walls. Nearly all columns in each story showed evidence of flexural damage. Floor framing was undamaged, except for minor cracking in standard beams near corner columns. MM VIII-IX.

17. Arabian Apartments, Long Beach 1933 (Green, 1933, pp. 560-562). Eight-story building with beam and joist framing and hollow tile curtain walls. First floor columns were cracked at intersection with second-story beams. Tile walls were badly cracked and broken out in the lower stories. MM VIII-IX.

18. Woodrow Wilson High School, Main Building, Long Beach 1933 (Davis, 1933, pp. 21-22). Reinforced concrete building with interior partitions of gypsum tile. Only damage to concrete elements occurred at the 80-foot tall tower. The bases of the concrete columns that supported the tower were badly damaged. Some cracking occurred at construction joints. Gypsum tile partitions were destroyed. MMI VIII-IX.

7.3.4 Loads and Load Paths

Gravity loads are transferred from the floor and roof diaphragms to the subframing which is supported by the concrete frames. The concrete frames may also support the weight of the infilled masonry walls and/or partitions.

Past earthquakes have shown that infilled masonry walls and partitions drastically alter the seismic response of this building type. In the elastic range, the stiffness of the infill causes the building to respond as a shear wall structure. Once cracks form along the boundary between the infill and the frame, the response is similar to that of a braced frame with the infill in compression acting as the diagonal elements. This response can lead to high concentration of shear stress in the concrete columns near the joints, which can lead to shear failures. If the masonry does not extend to the beam soffits, a short column condition may exist. As the stiffness of the masonry infill degrades, the concrete frames may begin to resist the lateral loads through frame action. Note that this scenario of response is often not that which was anticipated by the original designer. In many cases, the stiffness of the infilled walls was ignored and only the frame action of the steel elements was considered.

Typical floor dead weights depend on the diaphragm material. Concrete floors may weigh between 90 and 130 psf. Typical brick masonry weighs approximately 120 pcf. Roof live loads are typically taken as 20 psf, and floor live loads may range from 40 to 100 psf, depending on the occupancy. These live loads may be reduced for members that support large areas.

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 Section 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 $0.4\ R_W$ 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. They were not designed for in-plane loadings, which could produce distress and create a potential falling hazard if the diaphragm stresses are large. 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 diaphragm shear forces to be resisted by the clay tile arch floors. If they exceed 120 pounds per foot, then perform further investigations of the materials. Evaluate the potential for damage to cause materials to fall from the slab underside. Check for spalled joints, tie rod size, spacing and condition, steel beam condition, floor cracks, and loose soffit tiles. For more information on this form of construction, see Kidder (1906, 1900-1920) and Sweets (1906).

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".

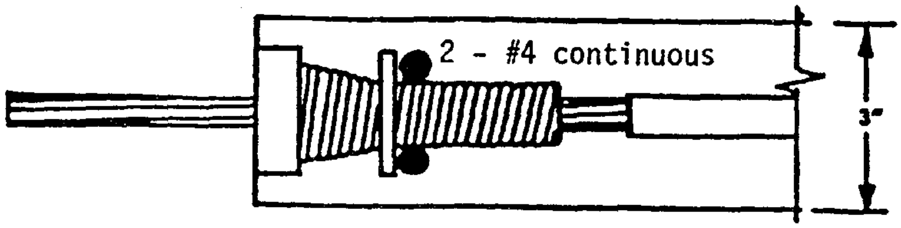
Recommended C/D Ratio: 1.0.

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.

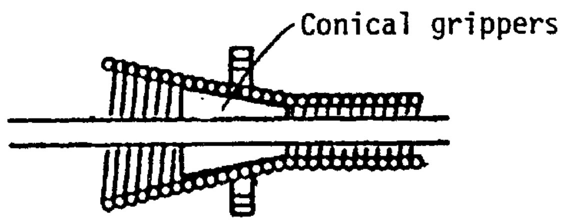
Concern: Poorly attached cladding and unbonded veneer courses can pose a falling hazard. The hazard created increases with the height above the building base.

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.



a. Section showing coil loop inside concrete deck.



b. Cone-shaped helix with conical grippers.

Coil loop anchors have been used in flat-slab apartment buildings, multi-level parking structures, and for post-tensioning of decks to make them watertight. Cables are normally unbonded, 3/8", 7/16", 1/2", or 5/8" in diameter, and ASTM 250 ksi or 270 ksi in strength. In this example, seven-wire cables are anchored individually with conical grippers housed in a cone-shaped helix of high-tensile wire.

Typical Coil Loop Anchor

FIGURE 7.1

SECTION 8

SEISMIC EVALUATION OF BUILDINGS WITH PRECAST CONCRETE ELEMENTS

8.1 Seismic Evaluation of Tilt-Up Buildings with Precast Bearing Wall Panels

8.1.1 Building Description

This building type is typically one or more stories in height. The roof diaphragm is generally composed of plywood sheathing, metal deck with or without concrete fill, or precast concrete elements. Floor diaphragms are typically metal deck with concrete fill, plywood, or precast concrete elements. Plywood sheathing is typically supported by wood or steel joists, beams, and columns. Metal deck diaphragms are generally supported by steel beams or open web joists and steel columns. Precast floor elements often span between wall elements. Exterior bearing walls are precast concrete panels that may or may not be interconnected.

8.1.2 Performance Characteristics

Tilt-up buildings with bearing wall panels have often performed well when the anchorage of the wall panels to the roof is detailed adequately. Poor performance has occurred when the anchorage details have insufficient strength and ductility. The following statements list some specific characteristics that these buildings may exhibit:

1. The wood ledger, which often provides the anchorage between the plywood diaphragm and the exterior precast panels, can fail in cross grain bending, possibly causing partial collapse of the roof and/or falling out of the wall.

2. Partial roof and/or floor collapse and/or wall falling out can also result from the perimeter nailing pulling out of the wood diaphragm or the wood ledger.

3. If the pilaster ties that secure the girder anchor bolts into the pilaster are insufficient, the girder can lose its vertical support, causing at least partial roof collapse.

4. Lack of continuous tension ties across diaphragms can cause roof and/or floor elements to separate and lose their vertical support.

5. Metal deck roofs may not be positively anchored to the walls in shear or tension, which can cause the walls and diaphragm to separate and fall.

6. Large eccentricities in connections of walls to metal roofs can lead to separation of the walls from the diaphragm.

7. Welded inserts used to connect adjacent panels may fail and permit the panels to separate.

8. Large openings in wall panels, especially at building corners, can distort and suffer damage. They behave essentially as frames, not shear walls.

9. Concrete parapets that are unreinforced and have large height-to-thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

10. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

11. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

8.1.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Warehouse, San Fernando 1971 (Murphy, 1973, p. 51). One-story, 131-1/2 feet x 276 feet. Tilt-up walls with cast-in-place closures, glu-lam beams and a panelized plywood roof. Interior pipe columns were founded on piles. Partial collapse occurred as a result of wood ledgers splitting and nails pulling out. Corbel anchorage to glu-lam also failed MM VIII-XI.

2. All Phase Color, San Fernando 1971 (Murphy, 1973, p. 55). Construction and damage similar to number 1 above. MM VIII-XI.

3. Vector Electronics, San Fernando 1971 (Murphy, 1973, p. 75). 180 feet x 240 feet, one-story building with tilt-up walls, cast-in-place closures, panelized plywood roof with glu-lam beams, and interior pipe columns. Partial collapse occurred as a result of split wood ledgers and nail pullout. Glu-lam beams also pulled off seats at walls. Damage was repairable. MM VIII-XI.

4. Elmendorf Warehouse, Anchorage 1964 (C&GS, 1967, p. 134). Building, 226 feet x 1000 feet plan (five units), had tilt-up walls with poured closures, steel interior framing, and a two-inch wood deck. Diaphragm had rod bracing, and walls were not connected to bracing for lateral forces. Several units collapsed.

5. Pepsi Cola Plant, Anchorage 1964 (C&GS, 1967, p. 20). Tilt-up wall building with precast T-beams without a topping slab, welded inserts for shear, and some concrete block partitions. Welded inserts at end panels broke, but building did not collapse.

6. National Bank of Alaska, Anchorage 1964 (C&GS, 1967, p. 23). Two-story building with tilt-up walls and precast T-beams with topping slab for second floor and roof. Glass exterior wall on one side. no building damage occurred.

7. Sandia Laboratories, Livermore, California 1980 (Degenkolb Associates, 1980, pp. 19-20). Concrete tilt-up building, 100 feet by 400 feet. Developed diagonal cracks from roof truss seats at tops of panels down to edge of panel due to style of panel to truss connection and out-of-plane forces. General architectural damage included wall cracking, displaced T-bar ceilings and lights, dislodged expansion joint covers, pounding and separation of elements of different stiffnesses, glass breakage, displaced books, scattered office contents, and movement of computer frames. MM VI.

Note: The following four buildings have cast-in-place rather than precast walls. These buildings were included here because the wood roof framing combined with the concrete walls (in this way) was a predecessor of modern tilt-up wall construction. Some differences in performance may occur.

8. Bank of Tehachapi, Tehachapi 1952 (SEAONC, 1952, p. 21; Degenkolb, 1955, p. 1289; Steinbrugge and Moran, 1954, p. 363). Cast-in-place concrete wall building with steel trusses, straight wood sheathing with rod bracing, and reinforced concrete block parapets. Some parapets fell, but otherwise undamaged. MM VIII-IX.

9. Catholic Youth Center, Tehachapi 1952 (Steinbrugge and Moran, 1954, p. 230; SEAONC, 1952, p. 49). Concrete wall building with wood floor and roof. No damage occurred except for minor cracking. MM VIII-IX.

10. Tehachapi State Prison for Women, Laundry Building and Cottages, Tehachapi 1952 (SEAONC, 1952, pp. 57-63; Steinbrugge and Moran, 1954, p. 227; Degenkolb, 1952, p. 10). Reinforced concrete wall building with wood roof and tile partitions. Not designed for earthquake. Major damage to timber, chimneys, and tile walls.
MM VIII-IX.

11. Morningside School, San Fernando 1971 (Murphy, 1973, p. 674). Pre-Field Act school later reinforced to meet the Field Act. Two-story concrete wall building with a concrete floor and wood roof. Some wall anchors and plywood shear wall were damaged. Also much plaster damage occurred. MM VIII-XI.

8.1.4 Loads and Load Paths

Gravity loads are transferred from the floor and roof diaphragms to the wood or steel joists and beams, or the open web joists. The major floor framing elements span to the exterior bearing walls or interior columns, which transfer the loads to the foundation.

Lateral loads are transferred from the diaphragms to the exterior bearing walls. The precast walls may act as single elements, or as a succession of individual panels, depending on the shear capacity of the connection between panels. The capacity of the wall anchorage for out-of-plane forces must be sufficient to prevent the walls from separating from the diaphragms.

Typical floor dead weights depend on the diaphragm material. Wood floor dead loads may range between 10 and 25 psf, whereas concrete diaphragms may weigh between 90 and 130 psf. Roof live loads are taken as typically 20 psf, and floor live loads may range from 40 to 100 psf, depending on the occupancy. These live loads may be reduced for members that support large areas.

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.

8.1.5.1 Evaluation of Materials

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. (See Figure 7.1 for Detail of Coil Loop Anchor.)

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 0.4 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.

8.2 Seismic Evaluation of Precast Concrete Frame and Concrete Shear Wall Buildings

8.2.1 Building Description

This building type contains floor and roof diaphragms typically composed of precast concrete elements with or without cast-in-place concrete topping slabs. The diaphragms are supported by precast concrete girders and columns. The girders often bear on column corbels. Closure strips between precast floor elements and beam column joints may be cast-in-place concrete. Welded steel inserts may be used to interconnect precast elements. Lateral loads are resisted by precast or cast-in-place concrete shear walls.

8.2.2 Performance Characteristics

Buildings with precast frames and concrete shear walls should perform well if the details used to connect the structural elements have sufficient strength and/or displacement capacity. In some cases, though, construction costs often led to the use of non-ductile connection details between the precast elements. The concrete shear walls should limit the interstory drift and thereby reduce the nonstructural damage. The following statements list some specific performance characteristics that these buildings may exhibit:

1. The interconnections between precast elements are often non-ductile and may fail at loads slightly above the design level. This can increase story drifts and may lead to separation of parts.

2. Inadequate bearing area and/or insufficient connection between precast floor elements and columns can lead to loss of the girder's vertical support.

3. Adjacent precast floor or roof diaphragm elements can separate due to inadequate connection.

4. Large seismic events can cause shear cracks and distress around openings. Spalling of concrete is possible if a sufficient number of large cycles occur.

5. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories. Shear walls that are not continuous to the base of the building may lead to column shear failures.

6. Wall construction joints can create planes of horizontal weakness that may lead to shear failure at a force level well below the expected capacity.

7. Insufficient chord steel and/or lap lengths can lead to wall bending failures.

8. Insufficient confining steel at chord lap locations may cause splices to fail before full development of the bars is attained.

9. Insufficient vertical stirrups in coupling beams results in a lack of shear resistance that can cause a loss of coupling action. Portions of the coupling beam can spall off and fall onto the area below.

10. Inadequate reinforcing of beam-column joints or location of beam bar splices at columns can lead to joint failures.

11. Foundation dowels that are insufficient to develop the capacity of the column steel above can lead to local column distress.

12. Offsets or eccentricities between girders and columns of exterior frames can cause unanticipated forces that may lead to distress in these frames.

13. Use of bent-up longitudinal reinforcing in beams as shear reinforcement can result in shear failure during load reversal.

14. Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent building are not at the same elevation, and where there are no shear walls parallel to the pounding forces located to directly resist these loads (i.e., at the end of the structure adjacent to the other building).

15. Concrete parapets that are unreinforced and have large height-tothickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

16. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

8.2.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Alaska Sales and Service, Anchorage 1964 (COGS, 1967, p. 48). Precast concrete elements with two cast-in-place shear walls in north-south direction. Precast T-beams had welded insert connections. Under construction and 4-inch topping slab on precast was not completed. General collapse.

2. Goodwill Industries, San Fernando 1971 (Murphy, 1973, p. One-story, 50 feet x 100 feet building with precast boomerang arches, precast walls, and roof slabs. Designed for Zone 3, 1946 UBC. Damage occurred at joints and welds. Building did not collapse, but was demolished after earthquake. It would have had a damage ratio of 35 percent if repaired. MM VIII-XI.

3. Fantoni Plant, Friuli, Italy 1976 (Stratta and Wyllie, 1919, pp. 32-34). Precast concrete tied arches having joint or hinge at ridge and double steel tie bars. Precast purlins between arches. Arches sit in pocket in cast-in-place longitudinal concrete frames. No positive moment or shear connections. Portion of building totally collapsed. Arches pulled off bearing connections. Newer complex had precast columns, girders, and long-span T-beam construction. Some hinging occurred in columns, and some precast beams shifted on top of the girders. MM VIII-IX.

4. Ledraplastic Toy Factory, Friuli, Italy 1976 (Stratta and Wyllie, 1979, pp. 35-38). Reinforced concrete longitudinal frames, spanning in transverse direction. Above the frames is s clerestory with small concrete mullions supporting a tied hollow tile and concrete joist system. No bracing in clerestory area and nominal reinforcing. Hinging occurred at the top and bottom of mullions as they attempted to resist the lateral forces of the roof structure. One pair of columns failed at their joint with the longitudinal beams. MM VIII-IX.

5. Snaidero Industrial Complex, Friuli, Italy 1976 (Stratta and Wyllie, 1979, pp. 39-41). Precast concrete construction with a sawtooth-shaped roof. There appeared to be a lack of positive connection between elements of the prefabricated roof. Roof collapsed in several areas. Considerable tilting of building resulted. Insulated metal exterior walls appeared to perform well. MM VIII-IX.

6. Solari Plant, Friuli, Italy 1976 (Stratta and Wyllie, 1979, pp. 42-45). One-story industrial structure with precast roof system on cast-in place columns and precast exterior walls. Wall panels fell off due to pullout of insert clips at bottom connections and shearing of bolt connections at top. Cantilever columns were spalled at base, indicating hinge action at bottom. North wall panels jumped 5 feet from original wall line indicating considerable ground motion. Evidence of movement of non-anchored interior machinery. MM VIII-IX.

7. Fanzutto Building, Friuli, Italy 1916 (Stratta and Wyllie, 1979, p. 46). Small industrial building with precast tied arches on reinforced concrete frame and precast panels. Shaking dislodged unanchored roof purlins; roof arches then rotated sideways and fell. Precast panels connected to roof by J-bolt hooked around pin. J-bolt not fastened to roof, merely grouted 2 inches into top of roof beam. Panels collapsed. MM VIII-IX.

8.2.4 Loads and Load Paths

Gravity loads are transferred from the precast floor elements to the precast concrete girders. Floor girders are supported by precast concrete columns and/or concrete shear walls that transfer the loads to the foundation.

Lateral loads are transferred from the floor diaphragms, composed of the precast elements and/or the topping slab, to the concrete shear walls or the moment resisting precast frames. The concrete shear walls may be cast-in-place or precast panels.

Typical floor dead weights may range from 90 to 130 psf. Live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy.

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. (See Figure 7.1 for Detail of Coil Loop Anchor.)

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 posttensioning 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.

Recommended C/D Ratio: 4.0.

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 0.4 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.

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.

SECTION 9

SEISMIC EVALUATION OF REINFORCED MASONRY BUILDINGS

9.1 Seismic Evaluation of Reinforced Masonry Bearing Wall Buildings With Diaphragms of Wood or Metal Deck With or Without Concrete Fill

9.1.1 Building Description

This building type typically includes floor and roof diaphragms composed of plywood or straight or diagonal sheathing. Metal deck with or without concrete fill may also be used for diaphragm elements. Wood roof and floors are typically supported by wood joists and beams that span to wood posts or small steel columns. Metal deck roof and floors are typically supported by steel beams and columns. The exterior masonry walls support the floors at the building perimeter. The walls may be reinforced brick or concrete block masonry. Wood ledgers may be used to anchor wood diaphragms to the exterior masonry walls.

9.1.2 Performance Characteristics

The performance of this building type has been similar to that of tilt-up wall buildings with precast bearing wall panels. The capacity of the wall anchorage to the diaphragms has proven to be a major cause of damage to this building type. When provided with good wall anchorage details, these buildings often perform well. The following statements list some specific characteristics that these buildings may exhibit:

1. The wood ledger, which often provides the anchorage between the plywood diaphragm and the masonry walls, can fail in cross grain bending, possibly causing partial collapse of the roof and/or falling out of the wall.

2. Partial roof collapse and/or wall falling out can also result from the perimeter nailing pulling out of the wood diaphragm or the wood ledger.

3. If the pilaster ties that secure the girder anchor bolts into the pilaster are insufficient, the girder can lose its vertical support causing at least partial roof collapse.

4. Lack of adequate tension ties across diaphragms can cause roof elements to separate and lose their vertical support.

5. Metal deck roofs may not be positively anchored to the walls in shear or tension, which can cause the walls and diaphragm to separate and fall.

6. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

7. Masonry parapets that are unreinforced and have large height-to-thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

8. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

9.1.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location year and location of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Boy's Market, San Fernando 1971 (Murphy, 1973, p. 113; Lew et al., p. 55). One-story building, about 160 feet x 200 feet in plan. Reinforced grouted brick walls with wood panelized roof on glu-lam beams and steel tube columns. Wood ledgers failed in cross grain bending and nail pullout. Partial collapse occurred. One glu-lam pulled off its pilaster. Building was razed after earthquake. MM VIII-XI.

2. Bell Metrics, San Fernando 1971 (Murphy, 1973, p. 63). One-story building, 96 feet x 137 feet in plan, with an all wood roof and reinforced grouted brick exterior walls. Wall pulled off walls by tearing nails through plywood. Twenty-five percent damage ratio. Replaced walls, improved anchors, and rebuilt. Corners of walls were also broken. MM VIII-XI.

3. Thrifty Mart, Valencia 1971 (Murphy, 1973, p. 91). One-story building, 170 feet x 177 feet in plan, with reinforced brick walls and a panelized plywood roof on tapered steel girders. Nearly open front. Collector bars pulled out and wood ledgers partly split. Damage ratio about 3 percent. Nine miles from fault. MM VI.

4. Pacoima Lutheran Medical Center, San Fernando 1971 (Murphy, 1973, p. 189). Two-story building with plywood floor and roof on wood joists, steel beams and tube columns, and reinforced grouted brick shear walls. Exterior of first floor is glass, and second floor had wood walls. Second floor also had steel tube Xbracing. Designed for 1963 L.A. Building code. No collapse, but major nonstructural damage and structural detail failures. Damage ratio 35 percent. &1&1 VIII-XI.

5. Safeway Store, Arvin 1952 (Steinbrugge and Moran, 1954, p. 232). Building has reinforced grouted brick masonry walls, wood roof, and rod bracing. No significant damage occurred. MM VIII-IX.

6. Soledad Canyon Elementary School, San Fernando 1971 (Murphy, 1973, p. 678). Building has reinforced grouted brick walls and a wood roof with steel beams. Five sites in district had a total damage of $30,325. MM VIII-XI.

7. Tehachapi High School Gymnasium, Tehachapi 1952 (SEAONC, 1952, pp. 9-10; Degenkolb, 1955, p. 1285; Steinbrugge and Moran, 1954, p. 366). Complete concrete frame building with reinforced concrete block walls, with steel trusses, wood plank roof and rod bracing. One rod broke at threads. MM VIII-IX.

8. Dentist Office, Tehachapi 1952 (SEAONC, 1952, p. 31). Reinforced concrete block wall, wood roof building. No damage occurred. MM VIII-IX.

9. Bank of America, Arvin 1952 (SEAONC, 1952, p. 73; Steinbrugge and Moran, 1954, p. 377). Building has reinforced concrete block walls, with stacked bond, and a wood roof with diagonal sheathing. Stacked bond walls had vertical cracks, but overall damage was slight. MM VIII.

10. Arvin High School, Arvin 1952 (SEAONC, 1952, p. 71; Degenkolb, 1955, p. 1288; Steinbrugge and Moran, 1954, p. 367). Concrete floor and roof are supported by grouted brick shear and bearing walls. Designed for earthquake under Field Act. Some damage to shear walls and interior contents occurred. MM VIII-IX.

11. Arvin High School Bus Garage, Arvin 1952 (SEAONC, 1952, p. 72; Steinbrugge and Moran, 1954, p. 375). Reinforced grouted brick wall building with wood roof, steel trusses and purlins, and rod bracing. Bracing failed but little other damage. MM VIII.

12. Hoblit Building, Anchorage 1964 (C&GS, 1967, p. 68). Two-story reinforced concrete block wall building with steel joists, and wood floor and roof. One side was essentially open. Minor damage occurred, mostly nonstructural in nature. Glass on open side was not broken.

13. Stones Liquor Store, San Fernando 1971 (Murphy, 1973, p. 45). Trapezoidal, One-Story, nearly open front, reinforced concrete block wall building with glu-lam beams, panelized plywood roof, and pipe columns. General collapse occurred. MM VIII-XI.

14. M & L Machine Shop, San Fernando 1971 (Murphy, 1973, p. 69. One-story, 90 feet x 142 feet reinforced concrete block wall building with glu-lam beams, panelized plywood roof, pilasters under glu-lams, and wood columns. Rear wall collapsed, wood ledgers split, nails pulled out, and block wall corners damaged. MM VIII-XI.

15. Grant's Department Store, Newhall, San Fernando 1971 (Lew et al., pp. 6-7; Murphy, 1973, p. 95). Reinforced masonry wall and wood floor building. Rear wall collapsed. Seven miles from epicenter. Wood ledgers split and nails pulled out. Damage ratio was about 15 percent. MM VI.

16. Wendell Machine Shop, San Fernando (Murphy, 1973, p. 83). Reinforced concrete block wall building with panelized plywood on glu-lam girders. Wood ledgers split and nails pulled out, and some glu-lams pulled away from wall pilasters. Corners of walls were also damaged. MM VIII-XI.

17. Bennett Industries, San Fernando 1971 (Murphy, 1973, p. 87). Reinforced concrete block wall building with panelized plywood roof on glu-lam beams. About 2 percent damage, material piled on walls. About 3/4 mile from fault rupture zone. MM VIII-XI.

18. Beekay Theatre, Tehachapi 1952 SEAONC, 1952, pp. 33-35; Degenkolb, 1955, p. 1290; Steinbrugge and Moran, 1954, p. 230). Reinforced concrete block wall building with wood roof, timber trusses, and straight sheathing with rod bracing. No damage occurred. MM VIII-IX.

19. Tehachapi Supply Company, Tehachapi 1952 (SEAONC, 1952, pp. 13-15; Degenkolb, 1952, p. 1285). Reinforced concrete block wall building with steel trusses, corrugated iron roofing, and concrete frame without bracing. Open front structure. Damage included broken windows at front. MM VIII-IX.

20. Simpson Motors, Arvin 1952 (SEAONC, 1952, p. 75; Degenkolb, 1952, p. 1292). Concrete block wall building with steel trusses, metal deck roof, and open front. Glass windows broke. Designed for earthquake. MM VIII.

21. Va. Media Agua School, San Juan, Argentina 1977 (Poland, in preparation, pp. 30-32). One hundred ten kilometers from epicentral region. Scalloped concrete plate roof on reinforced brick masonry bearing walls. Lateral system consisted of transverse full height, masonry shear walls (undamaged) and longitudinal, partial height masonry shear walls with full height columns. These columns failed in shear and bending. Roof deflections caused some damage to corners of masonry walls.

22. Music Building at Hipolito Vieytes School, San Juan, Argentina 1977 (Poland, in preparation, pp. 33-36). One-story, rectangular plan with two-level roof. Full height masonry pier in the front wall, columns spanning between the masonry walls and high roof on two sides, columns along rear interior wall. Having only one full height shear element, structure was highly eccentric, allowing for torsional forces that contributed to column failures and ultimately collapse at roof structure. (Other similar buildings on the site that had more balanced systems sustained no significant damage).

23. Bishop Elementary School, Coalinga, California 1983 (CDMG, 1983, pp. 39-40). Plywood roof over wood joists and steel beams with reinforced concrete block walls. Light cracking in concrete block walls where roof beams are supported. Ceiling damage also occurred. MM VIII.

9.1.4 Loads and Load Paths

Gravity loads are transferred from the floor and roof diaphragms to the wood or steel joists and beams. The major floor framing elements span to the exterior bearing walls or interior columns, which transfer the loads to the foundation.

Lateral loads are transferred from the floor diaphragms to the exterior masonry bearing walls. The capacity of the wall anchorage for out-of-plane forces must be sufficient to prevent the walls from separating from the diaphragms.

Typical floor dead weights depend on the diaphragm material. Wood or untopped metal deck floor dead loads may range from between 10 and 25 psf, whereas concrete topped metal deck may weigh between 70 and 110 psf. Roof live loads are typically taken as 20 psf, but floor live loads may range from 40 to 100 psf, depending on the occupancy. These live loads may be reduced for members which support large areas.

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-Δ 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 0.4 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 ACI 318 minimum value (ACI, 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.

Recommended 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.

9.2 Seismic Evaluation of Reinforced Masonry Bearing Wall Precast Concrete Diaphragm Buildings

9.2.1 Building Description

This building type includes floor and roof diaphragms that are typically composed of precast concrete elements such as planks, T-beams, or slabs. These diaphragms may or may not include a concrete topping slab. The precast slab elements are supported at the exterior by reinforced masonry walls. Interior support of the slabs may consist of steel or concrete frames, or masonry walls. The walls typically consist of either grouted brick or concrete block masonry.

9.2.2 Performance Characteristics

The performance of reinforced masonry bearing wall buildings with precast floor elements depends on the ability of the connections to tie the structural elements together and allow the structure to act as a unit when subjected to lateral forces. If the elements are well connected, the masonry walls should limit the story drift and thereby reduce the non-structural damage. The following statements list some specific performance characteristics that these buildings may exhibit:

1. Inadequate anchorage of precast floor elements into masonry bearing walls can cause the diaphragm members to pull away from the walls and collapse if adequate support length is not available. Generally, the support length is not adequate.

2. Adjacent precast diaphragm elements may separate if the connections do not possess adequate strength and/or ductility. Details with welded steel inserts are especially susceptible to such response.

3. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

4. Masonry parapets that are unreinforced and have large height-to-thickness ratios, or that are not anchored to the roof diaphragm, may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

5. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

9.2.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Chrysler Center, Anchorage 1964 (C&GS, 1967, p. 41). Reinforced concrete block wall building with precast T roof beams. Designed for Zone 3 of 1961 UBC. Open front structure with no collector bars to sidewalls parallel to front. Welded inserts for shear between T beams. Collapsed.

2. Western Radio and Telephone Building, Anchorage 1964 (C&GS, 1967, p. 26). Reinforced concrete block walls, precast T-beam building with fill. Collapsed.

3. Romig Junior High School, Anchorage 1964 (C&GS, 1967, pp. 22 and 229). Reinforced concrete block wall building with precast T-beams, and no topping slab for roof. Floor of gym was wood, and a lean-to roof had steel joists and metal deck. Distress occurred at supports of 90 feet span T-beams on block walls.

4. Golden State Community Health Center, San Fernando 1971 (Murphy, 1973, p. 199). Two-story reinforced concrete floor and roof building with reinforced grouted brick shear walls. Designed to meet the 1967 UBC. This well-detailed structure suffered little damage. MM VIII-XI.

9.2.4 Loads and Load Paths

Gravity loads are transferred from the precast floor elements to the exterior masonry walls and/or interior supporting elements, which may consist of either masonry walls or steel or precast T concrete frame elements. In narrow buildings (below approximately 75 feet) precast T-beam elements may span the entire width of the building.

Lateral loads are transferred from the precast floor elements to the reinforced masonry walls. These masonry walls may include interior walls in addition to those around the building perimeter. The walls may consist of reinforced brick or concrete block masonry. Adjacent precast floor elements are connected to provide proper diaphragm action.

Typical floor dead weights may range from 90 to 130 psf. Floor live loads are generally assumed to range from 40 to 100 psf, depending on the occupancy. Roof live load is typically taken as 20 psf. These live loads may be reduced for members that support large areas.

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-Δ 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.

Recommended C/D Ratio: 4.0.

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 0.005H. Verify that all vertical load carrying elements can maintain their load carrying ability under the expected drifts. Use 0.4 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 ACI 318 minimum value (ACI, 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.

SECTION 10

SEISMIC EVALUATION OF UNREINFORCED MASONRY BEARING WALL BUILDINGS

10.1 Building Description

This building type includes structural elements that vary, depending on the age and, to a lesser extent, the geographic location of the structure. Prior to 1900, the majority of floor and roof construction consisted of wood sheathing supported by wood subframing. Multiple layer sheathed floors may include one layer of boards that is laid diagonal to the subframing. Cast-in-place concrete floors were commonly used in large multi-story structures. These concrete floors may be supported by the unreinforced masonry walls and/or steel or concrete frames. Post-1950 unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. More recently, in regions of lower seismicity, these buildings may include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. Fired solid clay brick is the most common masonry unit employed for pre-1940 buildings. Natural or cut stone was also used for exterior walls. In many buildings of early construction, the exterior wythe may be joined to interior wythes only by the mortar placed in the collar joint. In other cases, different wythes may be tied together by using bricks laid with the long dimension across the collar joint (headers). Recent practice often leaves the collar joint free of mortar (cavity construction) with the bonding between wythes dependent on light gage metal ties. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties are usually less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can alter the building's seismic response by limiting both interstory drifts and diaphragm displacements.

10.2 Performance Characteristics

Unreinforced masonry bearing wall buildings have proven to present a significant hazard to life safety during relatively high intensity earthquakes. These hazards typically result from a lack of sufficient anchorage between the bearing walls and diaphragms, which in turn may result in a separation of parts and possibly collapse of walls and/or floors. The following statements list some specific performance characteristics that these buildings may exhibit:

1. Unreinforced masonry parapets or cornices extending above the roof level can fall.

2. Diaphragms that are insufficiently anchored can separate from the masonry walls, causing wall collapse. If the collapsed wall is a bearing wall, possible partial floor collapse can occur. Unanchored gable ends of masonry walls are especially susceptible to this problem.

3. Unbonded or inadequately tied exterior veneer courses can separate from the masonry wall and fall.

4. Beams that are supported on wall pilasters can pull off of their support due to lack of anchors and/or ties to provide confinement around anchors.

5. Walls that are adequately anchored to the floor diaphragm can exhibit large diagonal cracks in the piers due to in-plane loads. If the mortar joints are weak, as in lime mortar, the cracks follow the joints. These walls generally remain stable and continue to support the floors.

6. Steel beams that span across open storefronts can be highly stressed or even pull off their masonry pilasters due to large deflections.

7. Slender walls can fail perpendicular to their plane, fall onto streets, sidewalks, or adjacent property, and lead to partial roof collapse.

8. Unreinforced masonry parapets that have large height-to-thickness ratios or are not anchored to the roof diaphragm may constitute a falling hazard. The hazard posed by a parapet increases in direct proportion to its height above the building base.

9. Buildings with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., re-entrant corners) if separation joints or special reinforcing has not been provided.

10. Buildings with abrupt changes in lateral resistance have often performed poorly in earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the "soft" stories.

10.3 Examples of Building Performance

Included herein are short descriptions of the performance exhibited in past earthquakes by buildings of this classification. In some cases, an individual building may not exactly fit into the model building classification. In these cases, the building is listed with the model building that it most closely characterizes. This list also includes the location and year of the earthquake, the reference from which this information is summarized, the damage, any reported repair costs or damage ratios, and the approximate Modified Mercalli intensity in that area. This list is not intended to be all inclusive, but rather to demonstrate past performance and provide references for more information on this building type.

1. Lodge Hall, Tehachapi 1952 (SEAONC, 1952, pp. 26-27; Degenkolb, 1955, p. 1287). Not designed for earthquake, collapsed. MM VIII-IX.

2. Stores, Tehachapi 1952 (SEAONC, 1952, pp. 22, 23, 36; Steinbrugge and Moran, 1954, p. 231, 233-236; CDMG, 1955, pp. 261, 262). Many of these buildings collapsed. MM VIII-IX.

3. Tehachapi Hospital Tehachapi 1952 (SEAONC, 1952, p. 44). Building had hollow tile walls and partitions and wood roof. Not designed for earthquake. Badly damaged. MM VIII-IX.

4. Juanita Hotel, Tehachapi 1952 (SEAONC, 1952, p. 41; Degenkolb, 1955, p. 1283; CDMG, 1955, p. 246). Unreinforced brick walls, and wood roof and floor. Serious damage occurred, with one person killed. Not designed for earthquake. MM VIII-IX.

5. Auto Supply Store, Arvin 1952 (SEAONC, 1952, p. 76; CDMG, 1955, p. 216). Unreinforced brick walls, wood roof, open front. Not designed for earthquake. Sustained major damage. MM VIII.

6. Vineland School (old), Arvin 1952 (SEAONC, 1952, p. 135). Old building with unreinforced brick walls and wood roof. Not designed for earthquake. Major damage, later demolished. MM VIII.

7. Midnight Mission, San Fernando 1971 (Murphy, 1973, p. 651). In downtown L.A., this two-story brick building had a wood floor and roof. Parapet correction had been done. Part of front wall collapsed, killing one. All walls were cracked. Building demolished at loss of $600,000. MM VI.

8. Morningside School, San Fernando 1971 (Murphy, 1973, p. 674). Two-story unreinforced brick building with wood floors and roof. Cracks in walls and damaged floor ceilings. Did not collapse, but may have if shaking continued. Had to be demolished. MM VIII-XI.

9. Brick Bearing Wall House, Santa Rosa 1969 (Steinbrugge et al., 1970, p. 11). Brick walls cracked. MM VII-VIII.

10. Kress Store, San Fernando 1971 (Murphy, 1973, p. 643). Two-story brick building with wood floors, built in 1924. Bricks were laid in lime mortar. Damage severe, estimated $200,000 for repair, so it was demolished.

11. Three-Story Loft Building, San Fernando 1971 (Murphy, 1973, p. 645). In downtown L.A. Front wall parapet had been braced. Opposite wall lost parapet and part of wall. Beams and joists pulled apart from wall. Cost of repair: $75,000. MM VI.

12. Warehouse, Tehachapi 1952 (SEAONC, 1952, p. 54; Steinbrugge and Moran, 1954, p. 239). Structure included unreinforced concrete block walls, steel trusses, diagonal sheathing for wood roof, reinforced concrete frame. Gable walls fell out. MM VIII-IX.

13. Unreinforced Masonry Buildings (general), Coalinga, California 1983 (Degenkolb Associates, 1983). Many structural failures occurred. Fallen parapets, wall collapse, diagonal shear cracking. Inadequate tying of roof diaphragms to masonry walls resulted in many wall and diaphragm collapses. MM VIII.

14. Coalinga District Administration Building, Coalinga, California 1983 (CDMG, 1983, p. 39). One-story building with plywood roof supported by wood joists, steel beams, and concrete block walls. Some concrete blocks were slightly cracked at roof beams. Plywood ceiling panels were displaced. MM VIII.

15. Museum in the Park, San Francisco 1906 (State Earthquake Investigation Commission, 1969, Part 1, pp. 230-231). Wood frame with brick and plaster walls. Walls cracked badly and some portions fell. Hemispherical sandstone block arch had cracking. Several blocks moved out of place in columns made of sandstone blocks, backed with brick. MM VIII.

16. Lawrence Livermore Laboratory, Livermore, California 1980 (Degenkolb Associates, 1980, pp. 17-18). Office wing built in 1960. This building had concrete masonry walls with numerous cracks and one interior partition that was leaning slightly. Offset of 3 inches to 4 inches at one parapet. Masonry found to be ungrouted and unreinforced.

17. Academy of Sciences Building, San Francisco 1906 (USGS, 1907, pp. 31, 76). Six-story structure with cast-iron concrete-filled columns and reinforced concrete floor construction. No damage to columns by earthquake. Brick walls were badly cracked. MM VIII.

18. Appraisers' Warehouse (U.S. Custom House), San Francisco 1906 (USGS, 1907, pp. 32, 77-78). Four-story, heavy building with brick walls, slate roof. Practically undamaged by earthquake. Chimney left standing. Slight cracking in brick walls may have been due to pre-earthquake settlement. MM VIII-IX.

19. U.S. Mint, San Francisco 1906 (USGS, 1907, pp. 42, 95-96). Three-story structure built on substantial pile foundation. Bearing walls of heavy, solid brick faced with granite. Floors consisted of brick arches between steel beams, finished in cement. Cast-iron columns. No significant earthquake damage, probably due to good foundations and heavy walls. MM VIII.

20. Palace Hotel, San Francisco 1906 (USGS, 1907, pp. 97, 149). Very heavy brick walls (2 feet or more thick) laid in cement mortar, braced by many cross walls and partition walls. Every 3 or 4 feet in height there was reinforcement consisting of bands of iron in the brickwork, riveted together at their ends and crossings. The structure withstood earthquake forces, walls practically undamaged. MM VII.

21. Majestic Theater, San Francisco 1906 (USGS, 1907, p. 40). Roof trusses spanning 80 feet carried on 18-inch walls insufficiently reinforced by pilasters. Brick bearing walls collapsed. Roof trusses over stage collapsed. MM VIII.

22. Unreinforced Masonry Performance, Imperial Valley 1979. Damaged varied widely depending on building configuration, material strength, and quality of repairs of previous earthquake damage. Damage included cracking, failure of anchors, parapet collapse, and broken glass.

Note: The following discussion concerns the performance of adobe structures in past earthquakes. These buildings fall into the general category of unreinforced masonry, but should be treated separately.

23. Adobe House, Tehachapi 1952 (SEAONC, 1952, p. 12; Steinbrugge and Moran, 1954, p. 241; CDMG, 1955, p. 219). Badly damaged. MM VIII-IX.

24. Adobe Construction, Guatemala 1976 (Husid and Arias, 1976). Adobe by itself can never efficiently resist lateral loads. Because it is economically impossible to eliminate adobe construction from the affected area, it is necessary to improve it.

25. Adobe Construction (general), San Juan, Argentina 1977 (Poland, in preparation, pp. 18-22):

    a. Residential Adobe Construction: Exterior and interior bearing walls usually plastered on the inside and occasionally on the outside. Wood beam roofs supporting slab or reed/mud roof. This system provides very little diaphragm support to walls. Damage ranged from total destruction to partial collapse to little damage. Damage variations due to differences in site response and quality of construction.

    b. Larger Adobe Structures: Adobe walls used as non-bearing, exterior closure walls, with wood or steel trusses on masonry columns providing vertical roof support. One structure collapsed when the adobe walls failed to provide continuous lateral support for the roof structure and were not assisted by the masonry columns. Wood trusses were undamaged.

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.

10.4 Loads and Load Paths

Gravity loads are transferred from floor and roof wood sheathed diaphragms to joists that span between exterior masonry walls and interior partition walls. Concrete diaphragm buildings are supported by the exterior masonry walls and interior frames of either steel or concrete.

Lateral loads are transferred from the diaphragm elements to the exterior walls through the wall anchors. Interior partitions may contribute to the lateral force resisting system by limiting both interstory drift and diaphragm displacement. Wall anchors secure the wall to the diaphragm for perpendicular loads.

Typical floor dead weights depend on the diaphragm material. Wood floors may weigh between 15 and 40 psf, and floor live loads may range from 40 to 100 psf, depending on the occupancy. These live loads may be reduced for members that support large areas. Typical brick masonry weighs approximately 120 pcf.

10.5 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.

10.5.1 Rapid Evaluation of Shear Stress in Masonry Walls

Concern: Masonry 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 masonry walls should be performed in all evaluations of this building type in regions of high or moderate seismicity.

Procedure: General 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 5 psi, a more detailed evaluation of the structure should be performed. This evaluation should employ a more accurate estimation of the lateral loads using the procedures suggested in Section 4.4. Calculate the wall capacities using the provisions of Section 24 of the Uniform Building Code (ICBO, 1985) and compute Capacity/Demand ratios. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.5.

Recommended C/D Ratio: 1.0

10.5.2 Evaluation of Materials

Statement 10.5.1: 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 10.5.2: 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 10.5.3: 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 10.5.4: 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 10.5.5: 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.

10.5.3 Evaluation of Structural Elements

Statement 10.5.6: 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 10.5.7: 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 10.5.8: There are no significant strength discontinuities in any of the vertical lateral force resisting elements.

Concern: Weak 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-Δ stresses.

Recommended C/D Ratio: 0.4 R_w.

Statement 10.5.9: 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 0.4 R_w times maximum calculated drift for evaluation.

Statement 10.5.10: 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 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 10.5.11: 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 stiffnesses to be sure that no soft story condition exists.

Recommended C/D Ratio: 0.4 R_w

Statement 10.5.12: 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 away 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: Recommended the addition of floor-to-floor elements that will minimize the effects of pounding where it occurs.

Statement 10.5.13: Masonry walls are connected to the wood or metal deck diaphragms; for out-of-plane loads steel anchors or straps are embedded in the wall and attached to a diaphragm cross tie.

Concern: Wall anchorage connections that are composed of other than steel anchors or straps that are attached to diaphragm cross ties may not provide adequate capacity or ductility.

Procedure: Evaluate the Capacity/Demand ratio of the wall anchorage using the equivalent lateral force procedure and the F_p value given by Equation 4.12. Check the load path between the wall anchors and the diaphragm cross ties. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.4.

Recommended C/D Ratio: 0.4 R_w.

Statement 10.5.14: 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. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.4.

Recommended C/D Ratio: 4.0.

Statement 10.5.15: Gable ends of unreinforced masonry walls are anchored to all diaphragm levels.

Concern: Gable ends of masonry walls may fail out-of-plane if they are not anchored to all diaphragm levels.

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

Statement 10.5.16: There are no openings in the diaphragms larger than 8 feet that are adjacent to the exterior masonry walls.

Concern: Large openings adjacent to masonry walls limit the available perpendicular-to-wall bracing. Walls may not be provided with such out-of-plane capacity, or may be required to span the opening. These large openings may also reduce the capacity for load transfer between the walls and diaphragms.

Procedure: Calculate Capacity/Demand ratios for the perpendicular to wall bracing provided using an equivalent lateral force procedure. This value should be compared with the anchorage force, Fp given by Equation 4.12. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.4.

Recommended C/D Ratio: 1.0.

Statement 10.5.17: 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. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.3.

Recommended C/D Ratio: 1.0.

Statement 10.5.18: The height/thickness ratios of the 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. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.2.

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

Recommended C/D Ratio: 3

Statement 10.5.19: The height/thickness ratios of the top story wall panels in a multi-story building are less than 9.

Concern: The dynamic stability of the 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 buildings has crosswalls or concrete diaphragms, the allowable height/thickness ratio can be increased to 14. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.2.

Recommended C/D Ratio: 3

Statement 10.5.20: The height/thickness ratios of the wall panels in other stories of a multi-story building are less than 20.

Concern: The dynamic stability of the unreinforced masonry wall panels depends on their height/thickness ratio. 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. Review the applicability of Section 4.4.6 for the building being evaluated. If this section is applicable, use the procedure presented in Section 4.4.6.2.

Recommended C/D Ratio: 3

10.5.4 Evaluation of Foundations

Statement 10.5.21: 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 10.5.22: 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 10.5.23: 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 10.5.24: 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.

10.5.5 Evaluation of Non-Structural Elements

Statement 10.5.25: 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 10.5.26: 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.

Statement 10.5.27: There is no unreinforced masonry chimney that extends above the roof level.

Concern: Unreinforced masonry chimneys often collapsed.

Procedure: Evaluate the Capacity/Demand ratio of the chimney using the equivalent lateral force procedure and the requirements for F_p given in Equation 4.12. If the chimney is large enough in plan, it may be acceptable as unreinforced masonry.

Recommended C/D Ratio: 3

Statement 10.5.28: 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.

SECTION 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 (Section 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 (Sections 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. Similar concerns apply in regions of lower 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 R_w 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 Section 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

   (a) 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.

   (b) 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. Brick Veneer with Concrete Block Backup

   (a) The brick veneer is supported by shelf angles or other element at each floor level. (LS)

   (b) The brick veneer is adequately anchored to the backup at locations of through-wall flashing. (LS)

   (c) 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)

   (d) The concrete block backup qualifies as reinforced masonry (high seismicity only). (LS)

   (e) The concrete block backup is positively anchored to the structural frame at 4'-0" maximum. (LS)

   (f) 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)

   (g) All eccentricities in connections are accounted for. (LS)

   (h) Connections appear to be installed generally in accordance with the construction documents. (LS)

(i) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(j) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(k) There is no cracking in the brick veneer indicative of substantial structural distress. (LS)

(l) 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

(a) The brick veneer is supported by shelf angles or other elements at each floor level. (LS)

(b) The brick veneer is adequately anchored to the backup in the vicinity of locations of through-wall flashing. (LS)

(c) 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)

(d) 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)

(e) Corrugated brick ties are not used. (LS)

(f) All eccentricities in connections are accounted for. (LS)

(g) Connections appear to be installed generally in accordance with the construction documents.

(h) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(i) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(j) There is no cracking in the brick veneer indicative of substantial structural distress. (LS)

(k) 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)

(l) Mortar joints in the brick veneer are well filled, and material cannot be easily scraped out from the joints. (LS)

(m) Additional steel studs frame window and door openings. (LS)

(n) There is no visible corrosion of brick ties, tie screws, studs, or stud tracks. (LS)

(o) There is no visible deterioration of exterior sheathing. (LS)

(p) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

4. Precast Concrete

(a) There are at least two bearing connections for each wall panel. (LS)

(b) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(c) 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)

(d) 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)

(e) Where inserts are used in concrete connections, the inserts are welded to or hooked around reinforcing steel. (LS)

(f) All eccentricities in connections are accounted for. (LS)

(g) Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts. (LS)

(h) Connections appear to be installed generally in accordance with the construction documents. (LS)

(i) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(j) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

5. Thin Stone Veneer Panels

(a) There are at least two bearing connections for each wall panel. (LS)

(b) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(c) 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)

(d) 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)

(e) Where inserts are used in concrete connections, the inserts are welded to or hooked around reinforcing steel. (LS)

(f) All eccentricities in connections are accounted for. (LS)

(g) Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts. (LS)

(h) Connections appear to be installed generally in accordance with the construction documents. (LS)

(i) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(j) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(k) Stone anchorages are adequate for computed loads using $C_p = 0.75$ and $C/D = 4$. (LS)

(l) There are no visible cracks or weak veins in the stone. (LS)

6. Glass and Metal Curtainwall Panels

(a) There are at least two bearing connections for each curtain wall panel. (LS)

(b) There are at least four connections for each curtain wall panel capable of resisting out-of-plane forces. (LS)

(c) 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)

(d) 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)

(e) All eccentricities in connections are accounted for. (LS)

(f) Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts. (LS)

(g) Connections appear to be installed generally in accordance with the construction documents. (LS)

(h) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(i) Where inserts are used in concrete connections, the inserts are welded to or hooked around reinforcing steel. (LS)

7. Wood/Aggregate Panels

(a) There are at least two bearing connections for each wall panel. (LS)

(b) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(c) 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)

(d) 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)

(e) All eccentricities in connections are accounted for. (LS)

(f) Connections appear to be installed generally in accordance with the construction documents. (LS)

(g) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(h) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(i) Additional steel studs frame window and door openings. (LS)

(j) There is no visible corrosion of tie screws, studs, or stud tracks. (LS)

(k) There is no visible deterioration of exterior sheathing. (LS)

(l) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

(m) There is no visible deterioration of screws or wood at panel attachment points. (LS)

8. Stucco Finish on Lath Panels

(a) 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)

(b) 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)

(c) All eccentricities in connections are accounted for. (LS)

(d) Connections appear to be installed generally in accordance with the construction documents. (LS)

(e) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(f) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(g) Additional steel studs frame window and door openings. (LS)

(h) There is no visible corrosion of tie screws, studs, or stud tracks. (LS)

(i) There is no visible deterioration of exterior sheathing. (LS)

(j) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

9. <u>Composite Expanded Polystyrene and Stucco Panels</u>

(a) There are at least two bearing connections for each wall panel. (LS)

(b) There are at least four connections for each wall panel capable of resisting out-of-plane forces. (LS)

(c) 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)

(d) 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)

(e) Connections appear to be installed generally in accordance with the construction documents. (LS)

(f) Elements of cladding connections are not severely deteriorated or corroded. (LS)

(g) There are no signs of leakage inside the building that may indicate internal deterioration of the wall. (LS)

(h) Additional steel studs frame window and door openings. (LS)

(i) There is no visible corrosion of tie screws, studs, or stud tracks. (LS)

(j) There is no visible deterioration of exterior sheathing. (LS)

(k) Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

<u>11.2.11 Parapets, Cornices, Ornamentation, and Appendages</u>

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)

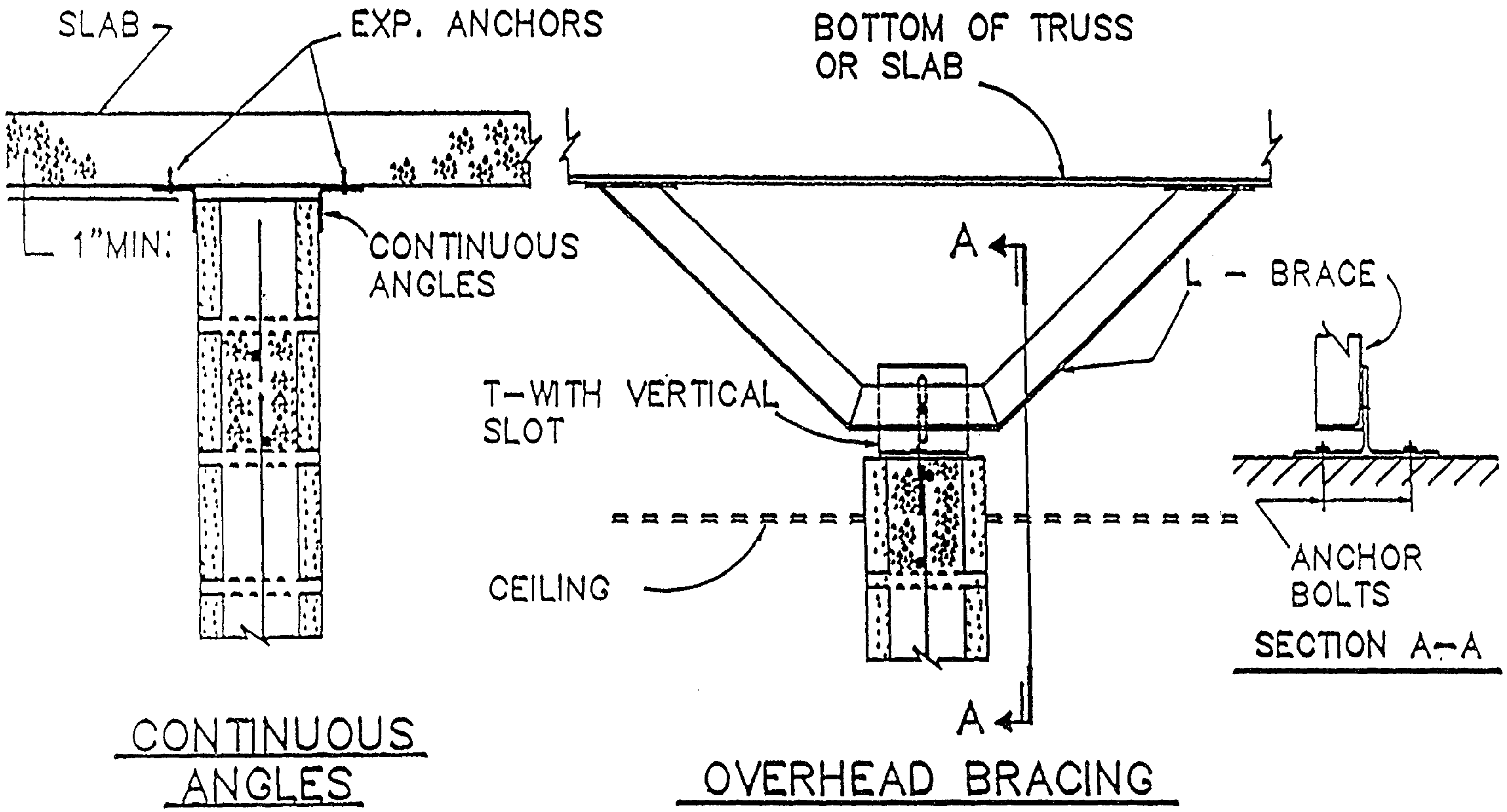


Figure 11.1 - Typical Details for Bracing Masonry Partition Walls

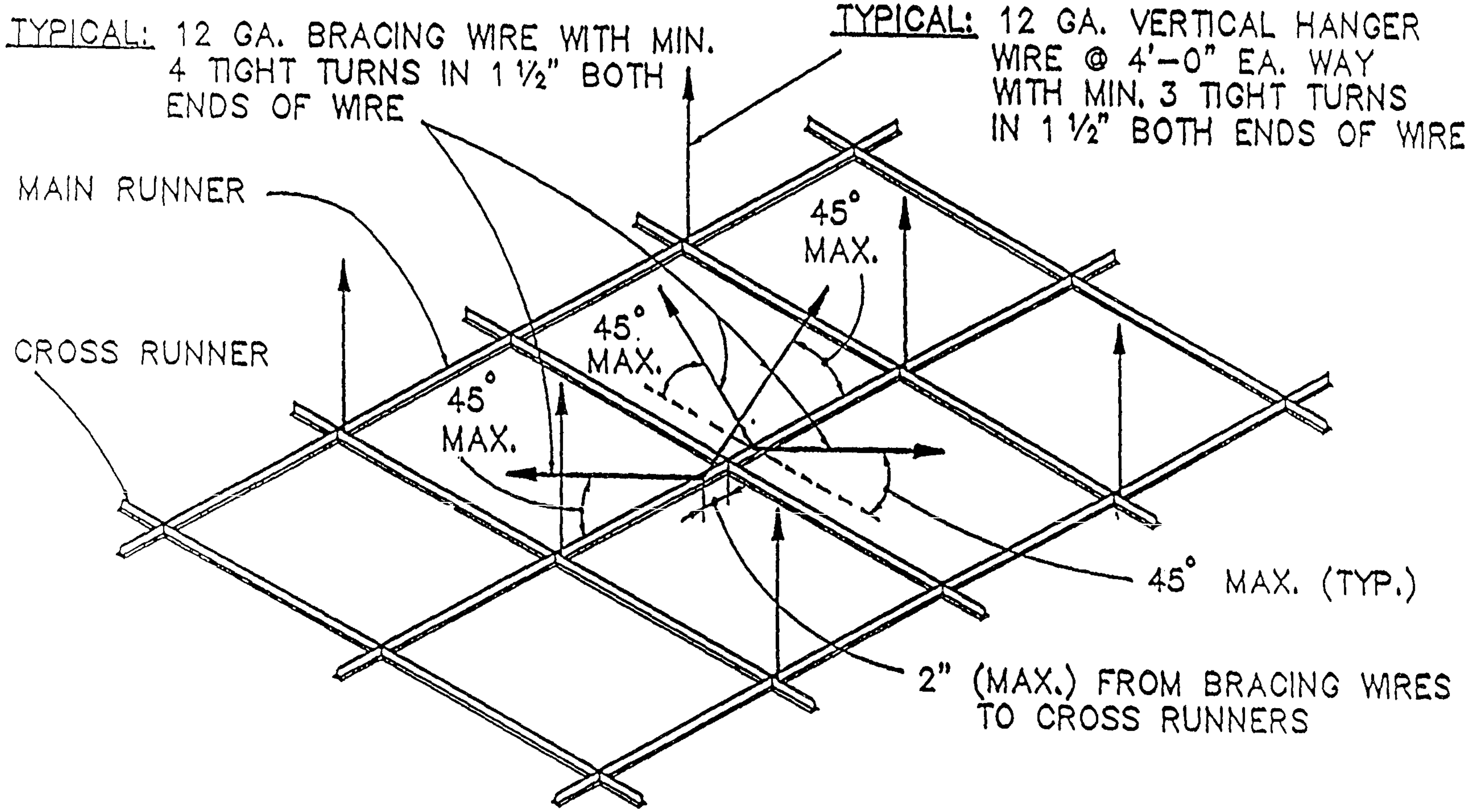


Figure 11.2 - Attachment of Acoustial Ceiling.Grid:

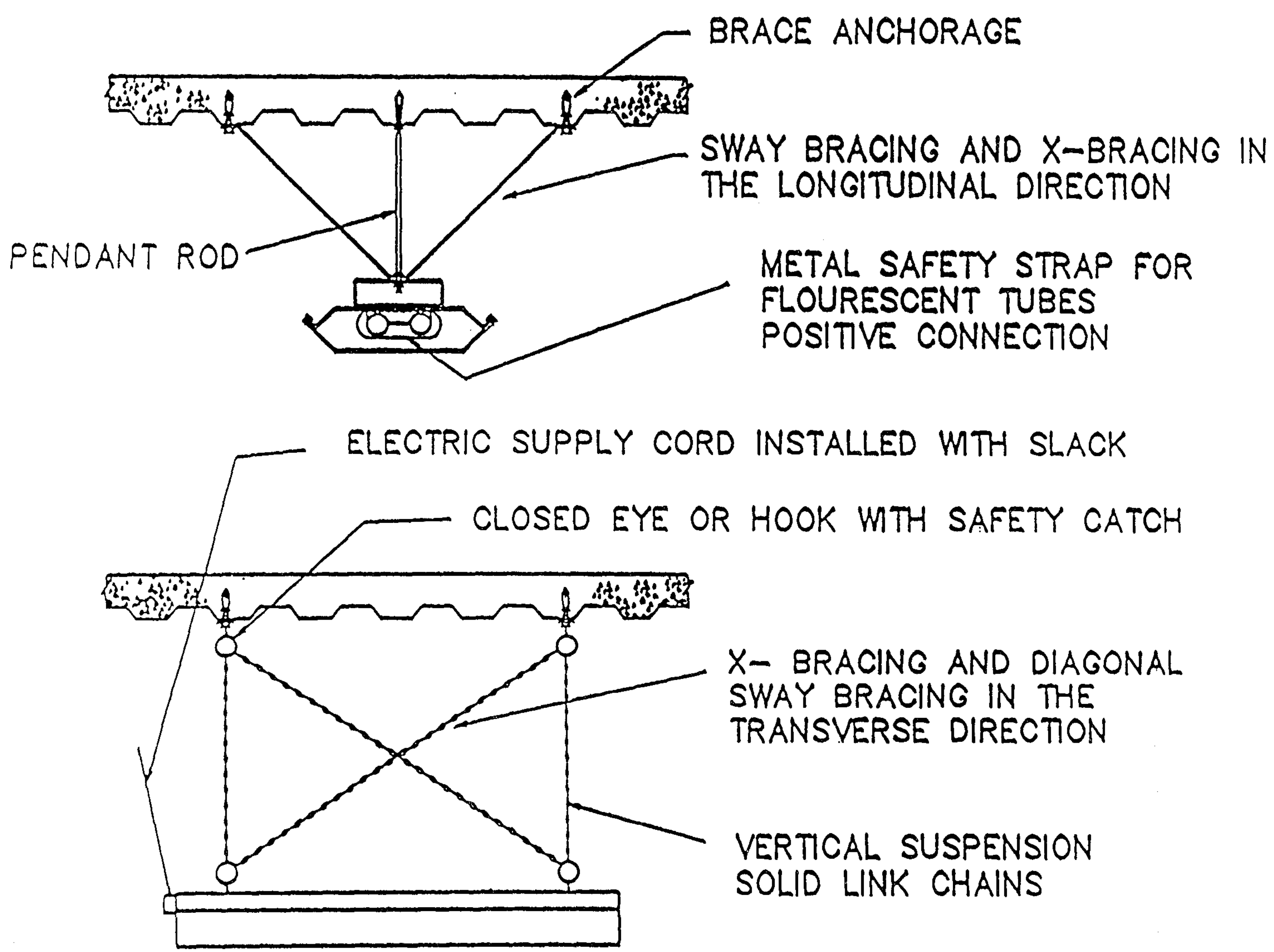


Figure 11.3 - Typical Anchorage of Suspended Lighting

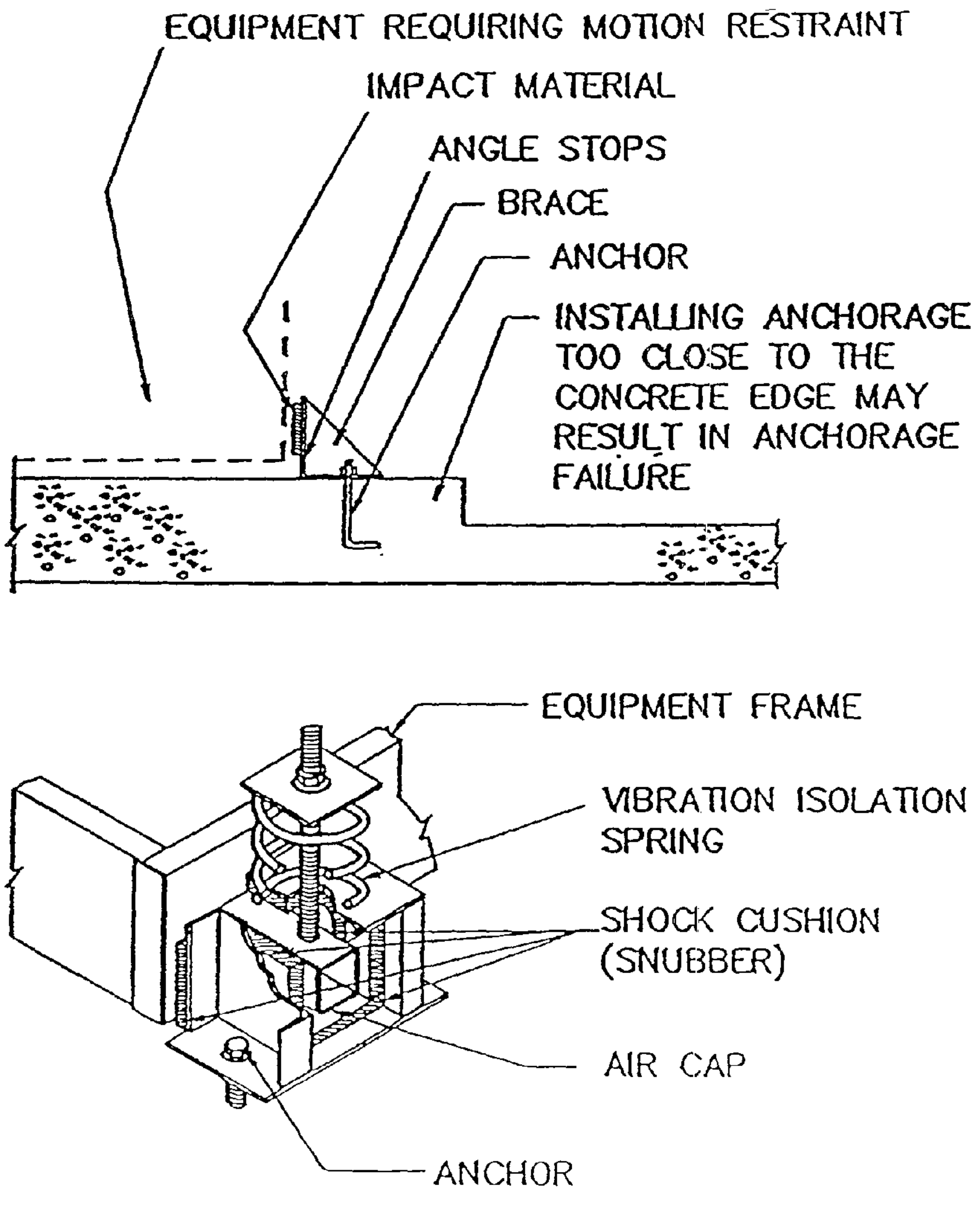


Figure 11.4 - Equipment Motion Restraint Systems

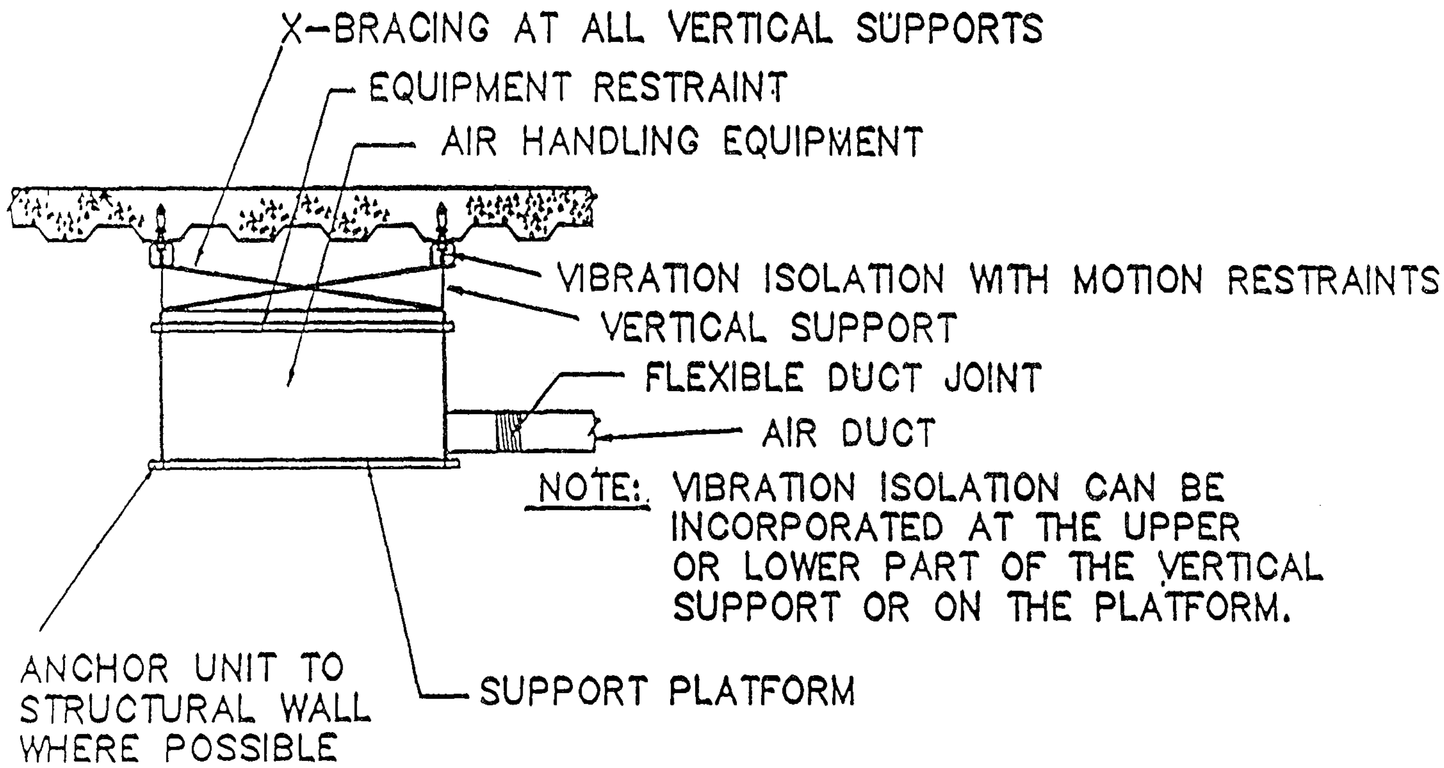


Figure 11.5 - Typical Anchorage of Air Handling Equipment

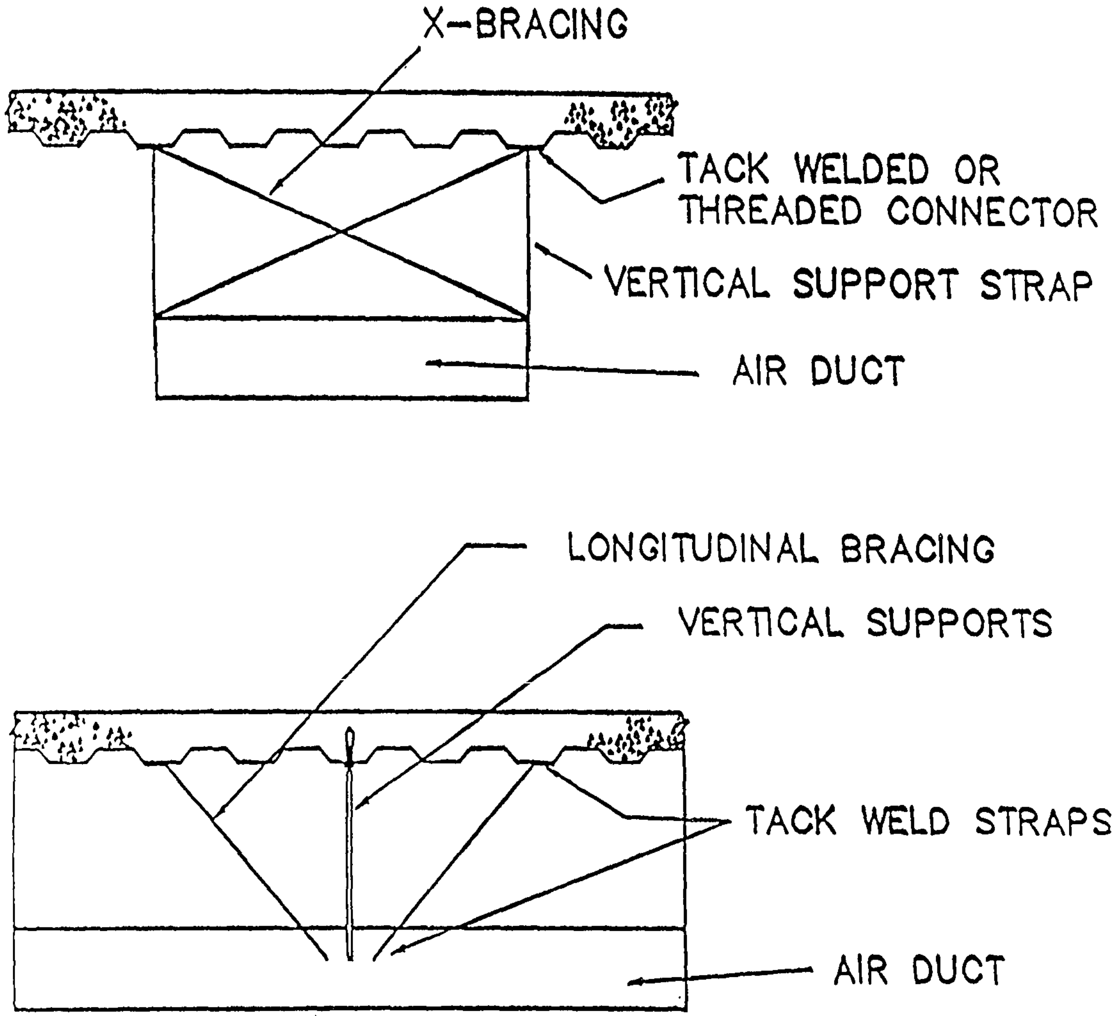


Figure 11.6 - Typical Suspension and Bracing of Ducts

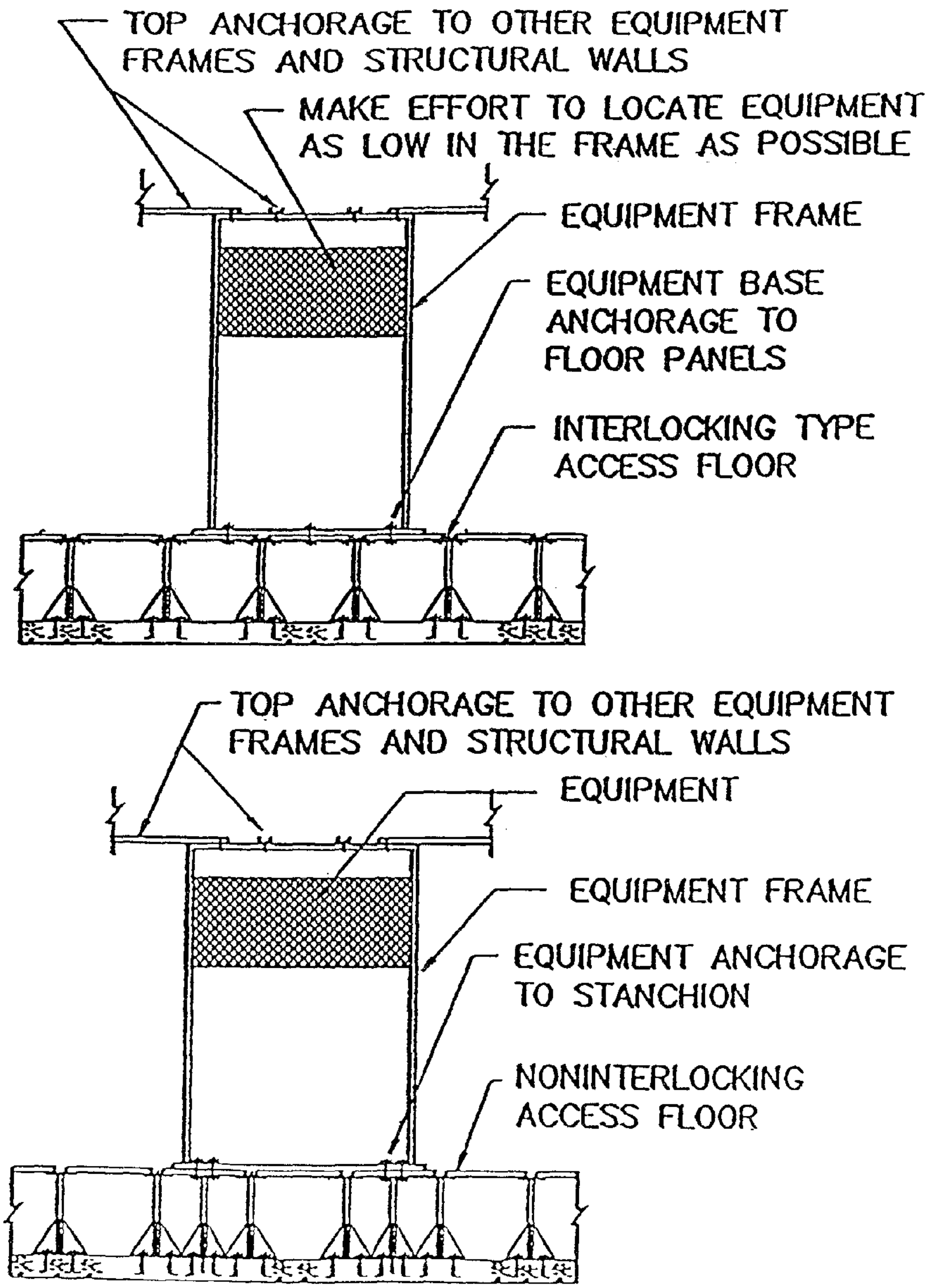
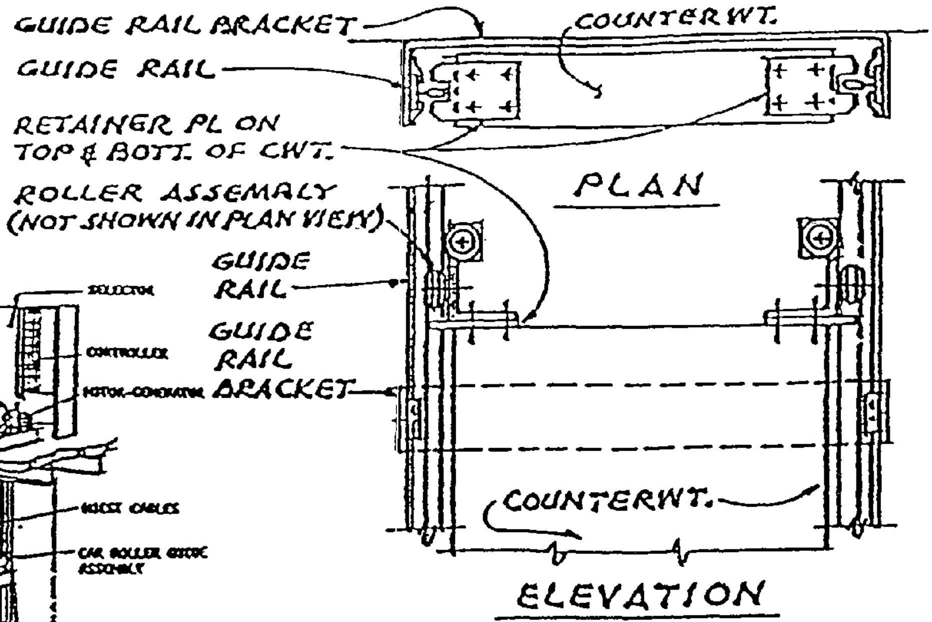
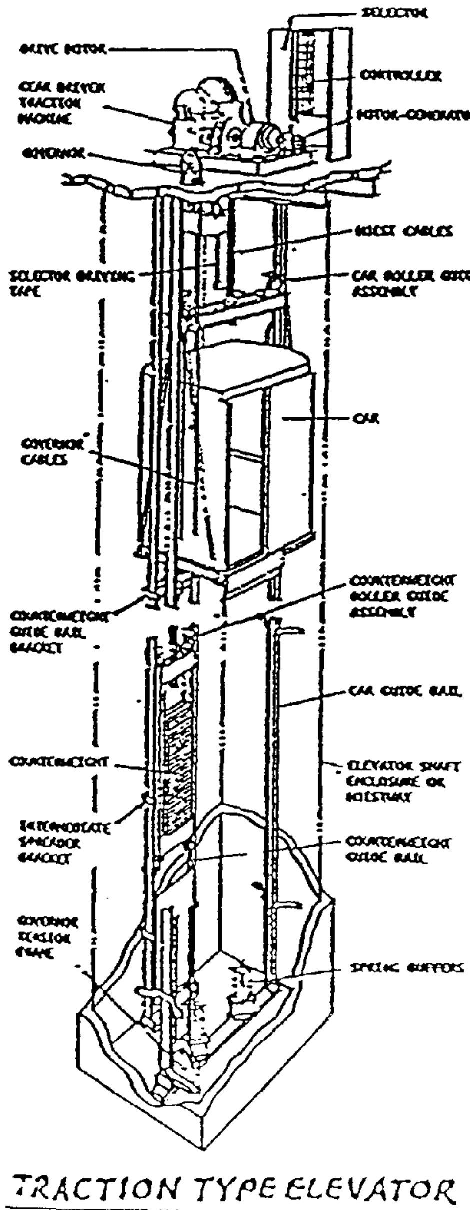
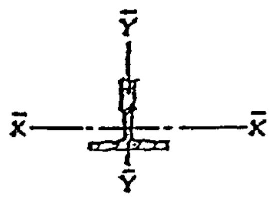


Figure 11.7 - Anchorage Details for Equipment Supported on Access Floors







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	1.60	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

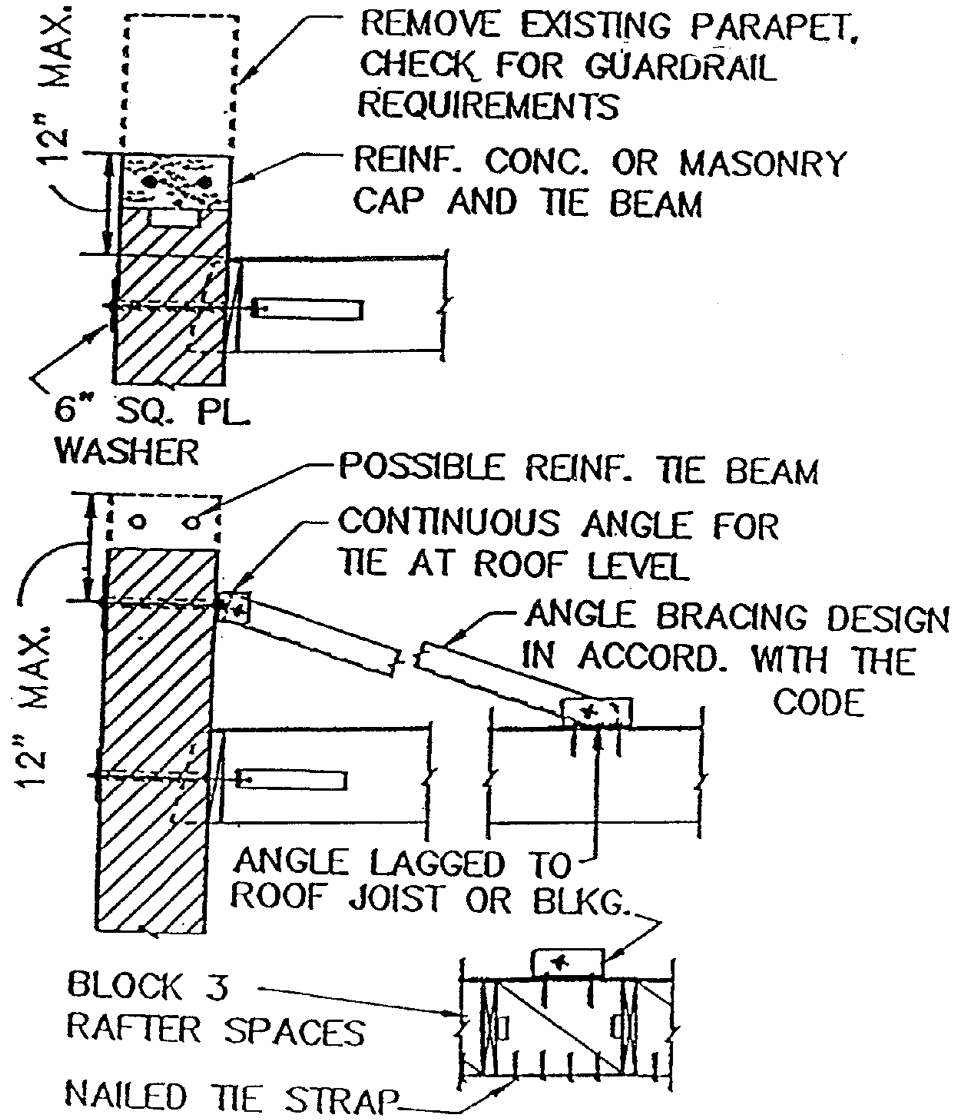


Figure 11.9 - Typical Parapet Anchorage Detail

SECTION 12

ILLUSTRATIVE EXAMPLES OF THE USE OF THE METHODOLOGY

This section illustrates the basic procedures to be followed in using the ATC-14 methodology. Four examples are included that describe the initial evaluation procedure. Each of the examples includes photographs and sketches of the structure, a completed field data sheet (Appendix C), building and nonstructural evaluation checklists (Appendix C), and a short narrative describing the results of the preliminary evaluation. The field data sheet has been designed to document basic information about the building and to help insure that none of the critical elements of the evaluation process are overlooked. The data sheet is formatted to document general information about the age and size of the building, construction data, a description of the lateral force resisting system, and the building classification (in terms of the ATC-14 model building types). The data sheet also contains a checklist pertaining to the critical elements of the evaluation process, and a section on earthquake damage potential, which is intended to allow for the evaluator's overall assessment of the potential for damage in a severe earthquake.

In each example, the items that require detailed evaluation are noted and discussed, but no specific calculations to obtain C/D ratios are presented. Examples 1 and 2 illustrate the rapid evaluation procedures for average wall shear stress and moment frame drift, respectively. The reader is directed to Section 4.1 and Figure 4.1 for a description of the steps involved in performing a seismic evaluation using the ATC-14 methodology.

All of these examples, which are taken directly from the original ATC-14 report are for buildings located in regions of high seismicity. As a result, the checklists in the examples are different from the low seismicity checklists in Appendix C of this document. Since the basic procedures are the same for areas of both high and low seismicity, these examples are still useful for understanding the proper use of this document.

12.1 Example 1, One-Story Concrete Frame Building with Reinforced Masonry Walls

The first example illustrating application of the ATC-14 methodology involves a one-story building with concrete frames and reinforced masonry infilled walls (Figs. 12.1 and 12.2). A plan view showing locations of the nonbearing concrete-block partitions, an exterior elevation, a longitudinal section of the building, and a typical section of the exterior walls are shown in Figures 12.3, 12.4, 12.5, and 12.6, respectively. Because this structure does not fit directly into any of the model building types, a multiple model building type designation is employed. In this case, the evaluation combines the procedures for Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry (Section 7.3) and for Reinforced Masonry Wall Buildings (Section 9.1).

Figure 12.7 shows a completed ATC-14 Field Data Sheet, which summarizes critical information about the building's characteristics, documents that all checklists and other critical assessments have been completed, and addresses the issue of earthquake damage potential. Structural and nonstructural evaluation checklist forms are provided in Figures 12.8 thru 12.10. Because this building has a multiple building type designation, the evaluation checklist forms for both model building types should be completed during the initial site visit and the subsequent examination of all available structural drawings. Since the structure being evaluated does not have a single building designation, some of the statements in the checklists may not be applicable. Statements that are not applicable (denoted as NA in data sheet) should be ignored during the evaluation process.

After completing the field survey, the initial step in the evaluation is a rapid check of the average shear stress in the masonry walls. The procedures suggested in Sections 4.4.2 and 7.3.6.1 provide a rough estimate of the wall shear stress. If the average shear stress exceeds the allowable limit, a more refined analysis of the wall loads and capacities should be performed. The application of this rapid evaluation procedure to the Example 1 building (Fig. 12.11) indicates that the infilled walls may be highly stressed. The evaluator would be required, therefore, to analyze the wall stresses more closely using the equivalent lateral force procedure suggested in Section 4.4.3.

The completed field-data sheets (Fig. 12.7), building evaluation checklists (Figs. 12.8 and 12.9) and nonstructural evaluation checklist (Fig. 12.10) are intended to call attention to any items that need detailed evaluation. For this building, the only detailed evaluation required for a structural item arises from the fact that the height/thickness ratio of the masonry walls exceeds the suggested limit of 14. The detailed procedure suggested in Section 7.3.6 for this structural element recommends that the out-of-plane stability of the panels be evaluated. A C/D ratio of 3 is recommended because of the lack of ductility anticipated for the response of this element.

In addition to the items flagged in the structural evaluation checklist, several items did not meet the nonstructural evaluation checklist statements, that is, the.answer to several statements was "false." Of the statements for which the response was "false," several have a LS designation, which indicates this condition could constitute a life-safety hazard. All items that do not pass the checklist statements should receive further consideration, especially any that could constitute a life-safety hazard.

12.2 Example 2, Five-Story Steel Moment Frame Building

The second example illustrating application of the ATC-14 methodology involves a five-story steel moment-frame building (Fig. 12.12). A typical floor plan, typical transverse moment frame, and typical longitudinal moment frame are shown in Figure 12.13, 12.14, and 2.15, respectively. This structure falls under Building Classification 6.1 and, because it is in an area of high seismicity, the applicable evaluation procedures begin with Section 6.1.6 of ATC-14.

A completed ATC-14 Field Data Sheet is provided in Figure 12.16 and structural and nonstructural evaluation checklists are provided in Figures 12.17 and 12.18, respectively. The information needed to complete the forms is obtained from both the field survey and the structural drawings.

As indicated in the structural evaluation checklist, a rapid estimation of building drift is required for this building type. Sections 6.1.6.1 and 4.4.2 present the procedure suggested for rapid evaluation of story drift. In order to use this procedure, the structural drawings must be studied and the lateral load resisting elements identified. In this example, there are four moment resisting steel frames of equal stiffness in each direction.

Calculations for the rapid evaluation of drift are provided in Figure 2.19. Initially, dead loads at the various levels are determined (Fig. 2.19, steps 1,2). Using Equations 4.1 and 4.2, base shear and story shears, respectively, are then determined (Fig. 2.19, steps 3,4), and the building model is established (Fig. 2.19, step 5). Once the shears have been obtained, the equation in Section 6.1.6.1 can be used to calculate the drift at any level. The quantities Kb and Kc (in this equation) are beam and column stiffnesses, respectively, and are simply I/L for each element. For this example, all of the beams are the same size as are all of the columns. If, however, different-size members are used in the same frame, an average I should be used. Similarly, if the member lengths in the same frame differ, an average L should be used. V_c is the average shear in each column of a frame. If all frames are equally stiff and are symmetrically distributed around the center of mass, V_c for a particular column is simply the total story shear divided by the number of columns. If the frame stiffnesses vary, however, V_c should be calculated considering the relative rigidities of the frame elements. In the case of the Example 2 building (Fig. 2.19, step 6) the story drift was calculated to be .0064, which is in excess of the .005 limit. As a result, a full frame analysis, using the Equivalent Lateral Force Procedure suggested in Section 4.4.3, is necessary.

Calculations to determine lateral loads using the recommended Equivalent Lateral Force Procedure are shown in Fig. 2.20. Lateral forces generated using this procedure are much smaller than those from the Rapid Analysis Procedure. Using a stiffness analysis computer program, the first-story drift for the Equivalent-Lateral-Force-Procedure loads was calculated to be .003, which is less than .005 limit.

The ATC-14 procedure for rapid drift estimation was intentionally designed to be conservative to insure that any building that is potentially too flexible will be required to undergo a more rigorous analysis. As a result, any building that does pass the rapid drift evaluation is considered to be well within the acceptable drift range. For this example, there appeared to be a problem with drift, but further analysis indicated the drift range to be acceptable.

12.3 Example 3, Nine-Story Reinforced Concrete Shear Wall Building

The third example illustrating application of the ATC-14 methodology involves a nine-story reinforced concrete shear-wall building (Figure 12.21). A typical floor plan, longitudinal elevation, and transverse elevation are shown in Figures 12.22, 12.23, and 12.24, respectively. This structure falls under Building Classification 7.2, and because it is in an area of high seismicity, the applicable evaluation procedures begin with Section 7.2.6 of ATC-14.

The Field Data Sheet (Figure 12.25) was completed following a visit to the site and with the aid of the original structural drawings. The site visit and structural drawings also provided the information necessary to complete the structural and nonstructural evaluation checklist statements for this structure (Figures 12.26 and 12.27, respectively).

As indicated in the structural evaluation checklist, the first step in the evaluation of shear wall structures is a rapid check of the wall shear stresses (as recommended in Section 7.2.6.1). Because the rapid evaluation procedure presented in Section 4.4.2 applies for structures less than or equal to six stories and the structure under consideration is nine stories, the wall shear and bending stresses should be evaluated using the Equivalent Lateral Force Procedure of Section 4.4.3.

The checklist statements identified a number of items that require further consideration or more detailed evaluation. Following are discussions regarding specific checklist statements:

Statement 7.2.6.2 (Deterioration of Rebar). Some spalled and exposed reinforcing was evident on the exterior concrete walls. Because this deterioration was not considered severe enough to materially reduce the capacity of these elements, no further evaluation of these elements, no further evaluation was deemed to be necessary. A recommendation to prevent further deterioration, however, would be appropriate.

Statement 7.2.6.6 (Torsion). The location and configuration of the transverse shear walls results in a large eccentricity between the centers of mass and rigidity. This is also the case for the longitudinal direction, since the frame-shear-wall system on Line A is more rigid than the frame on Line C. Because of these eccentricities, a rigidity analysis should be performed to determine the load distribution to each of the walls. A rigid diaphragm analysis that includes the shear and bending stiffness properties of the vertical elements would be appropriate. If applicable, the flexibility of the foundation system could also be included in the model. The load distribution from this analysis will also be used in addressing other checklist items that require detailed evaluation.

Statement 7.2.6.11 (Wall H/D Ratios). There are shear walls with H/D ratios greater than 4 that need to be evaluated to determine overturning capacity. In determining the overturning capacity of these elements, all available resistance, such as all dead load that can be activated by the wall, should be used.

Statement 7.2.6.14 (Shear Walls Adjacent to Diaphragm Openings). The transverse walls at the interior stair do not meet the requirements of this statement. The shear transfer capacity between the floor diaphragm and these walls should be compared with the load demands to the appropriate walls, which were generated by the lateral force analysis.

Statement 7.2.6.15 (Boundary Elements in Shear Walls). None of the shear walls have ductile details (spirals or closely spaced ties) for the boundary elements. Without such details, the boundary elements are believed to have limited ductility and, therefore, the recommended C/D ratio is 0.2 R_w. The capacity of these elements can be estimated using the resource document suggested in Section 4.4.1. The demand is determined from the results of the lateral force analysis.

12.4 Example 4, One-Story Wood Frame Building

The fourth example involves a large one-story commercial wood-frame building (Fig. 12.28). As indicated in the roof plan view (Fig. 12.29), the structure consists of five separate buildings built with different roof framing schemes. A view of the truss framing system for Building "A" is shown in Figure 12.30; sections of the various roof framing truss systems are shown in Figure 12.31. Because the overall structure consists of five separate buildings, all five of the buildings should be evaluated separately, including any possible interaction that could occur. The overall structure is classified as a Type 2 wood structure and, because it is in an area of high seismicity, the applicable evaluation procedures begin with Section 5.6 of ATC-14.

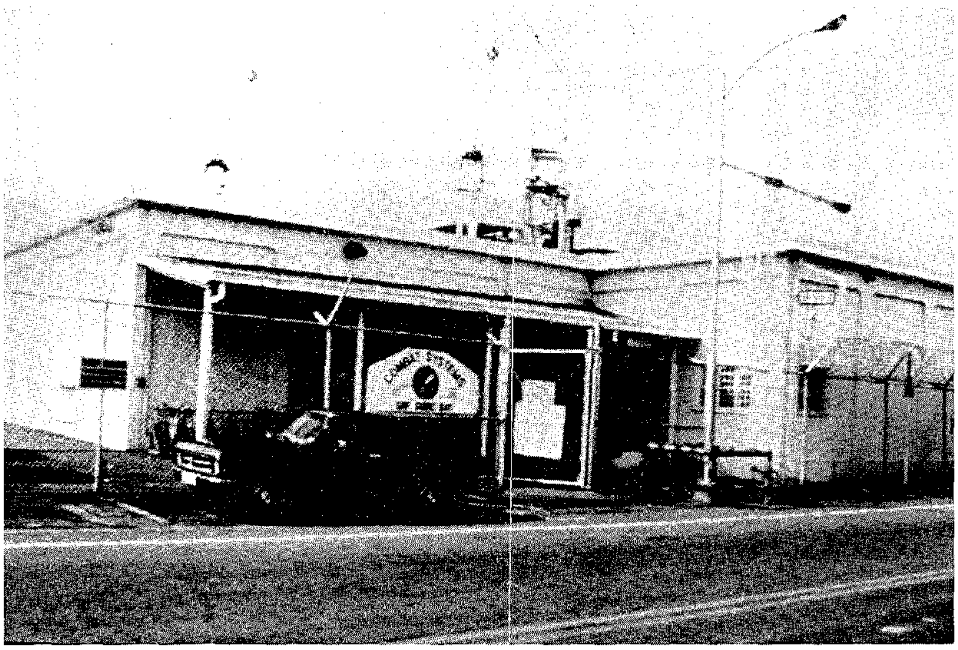
A completed ATC-14 Field Data Sheet is provided in Figure 12.32 and the structural evaluation checklist is shown in Figure 12.33. The information needed to complete the forms was obtained during the site visit, which was particularly important in this case because no drawings of the structure were available. As a result, numerous sketches of the structural elements were required, including the basic floor plan, truss configurations and sizes, and typical connection details. Major building dimensions were also taken during the site visit.

The first consideration in evaluating the seismic resistance of this structure is the rapid evaluation of the shear stress in the lateral force resisting elements. Building "A" has concrete walls on three sides, but no lateral load resisting element on the fourth side. None of the other four structures have any lateral force resistance in the transverse direction, as they are all open to allow passage through the building. The longitudinal direction has light metal siding that is not well attached to the wall framing or the roof. The building, therefore, does not have a complete lateral force resisting system that can be evaluated by the rapid shear stress check. This obviously constitutes a significant hazard that must be brought to the owner's attention. In addition, the unreinforced block wall and parapet between Buildings "D" and "E" could constitute a falling hazard that should be evaluated.

Because the structure does not have any wood shear walls or other vertical lateral force resisting elements, not all of the checklist statements are applicable to this building. The first checklist item of concern deals with serious deterioration of the structural elements. One of the roof trusses in Building "D" has such bad deterioration at the center post that the truss has dropped 6 inches. In fact, the wood post and the truss chords are so badly rotted that their capacity is questionable. The owner should be informed of these problems.

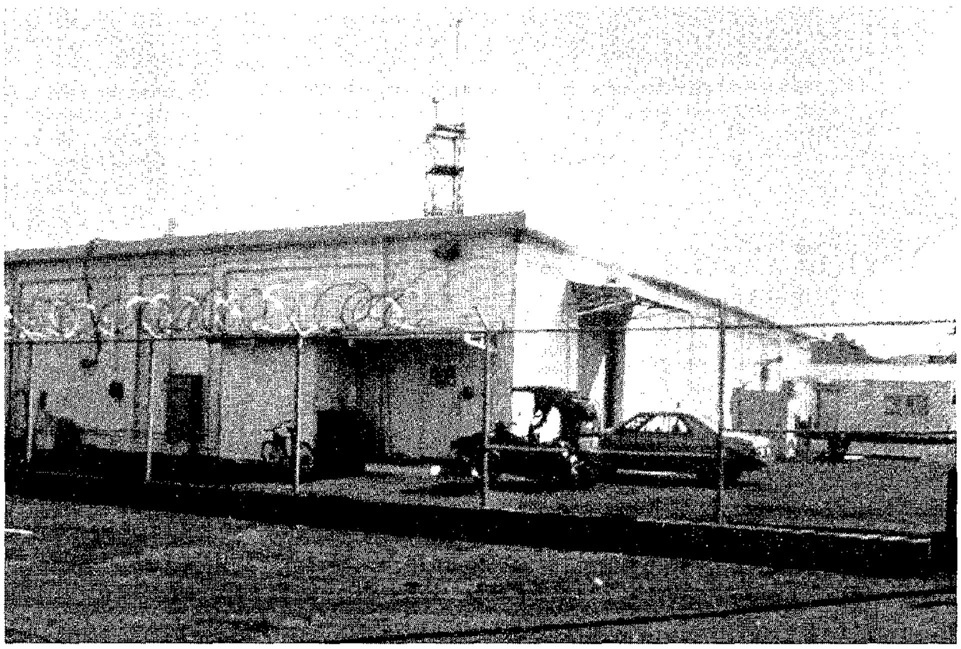
The large span/depth ratio of the straight sheathed diaphragm in Buildings "B" and "E" are also of concern (Statement 5.6.10). The ability of these diaphragms to span between the walls should be investigated using the suggested procedure in Section 4.4.6. Note that this procedure assumes that no transverse shear walls exist between the end walls. If a shear wall were added at the center of the buildings, the span/depth ratio for these diaphragms would be acceptable. The analysis procedures of Section 4.4.6 should also be used to evaluate the allowable C/D ratios for the diaphragms of all five buildings.

Since this example building is presently vacant and the entire structure is exposed . (i.e., there are no nonstructural elements), there is no need to complete the nonstructural checklist.



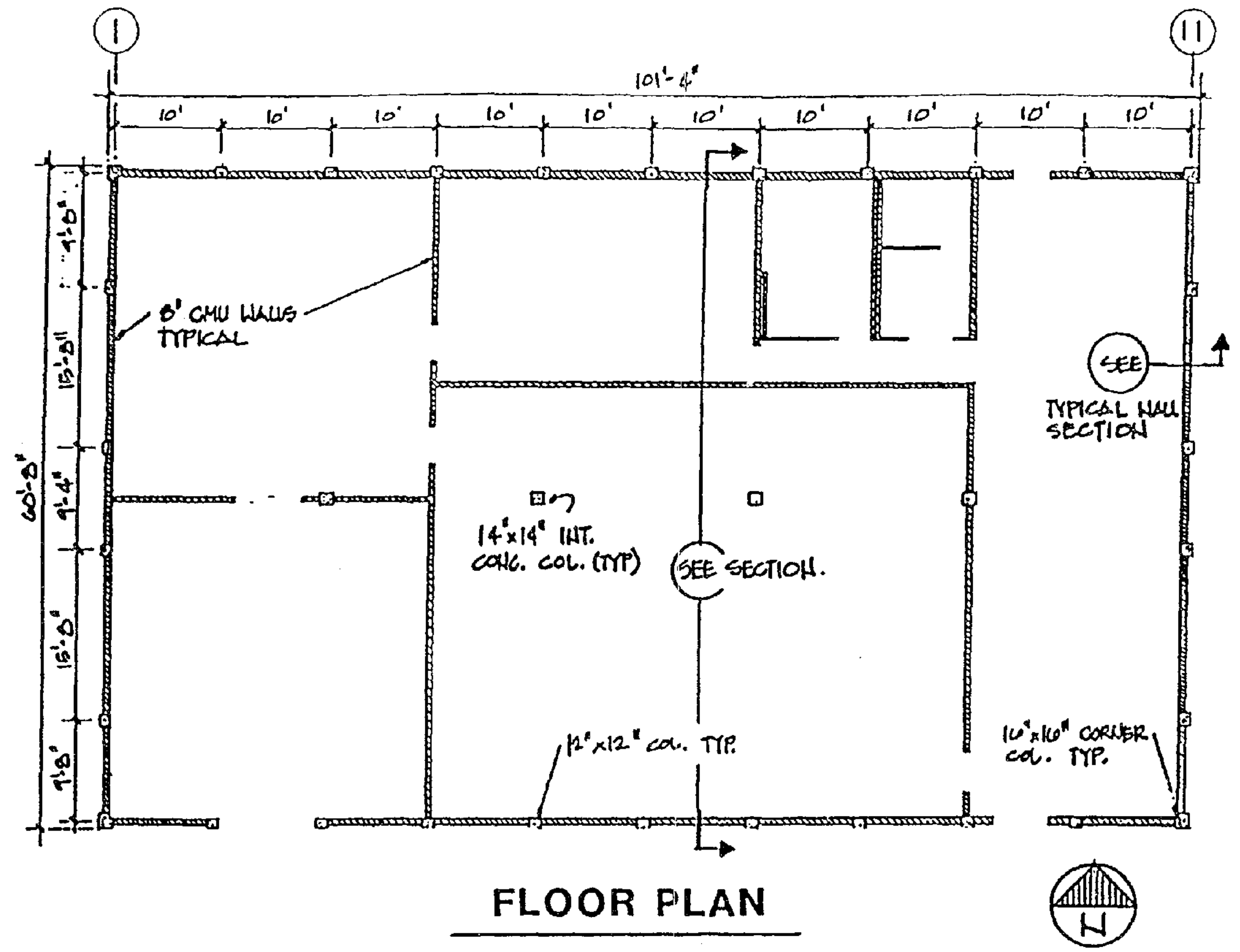
Exterior View, Example 1 Building.

FIGURE 12.1



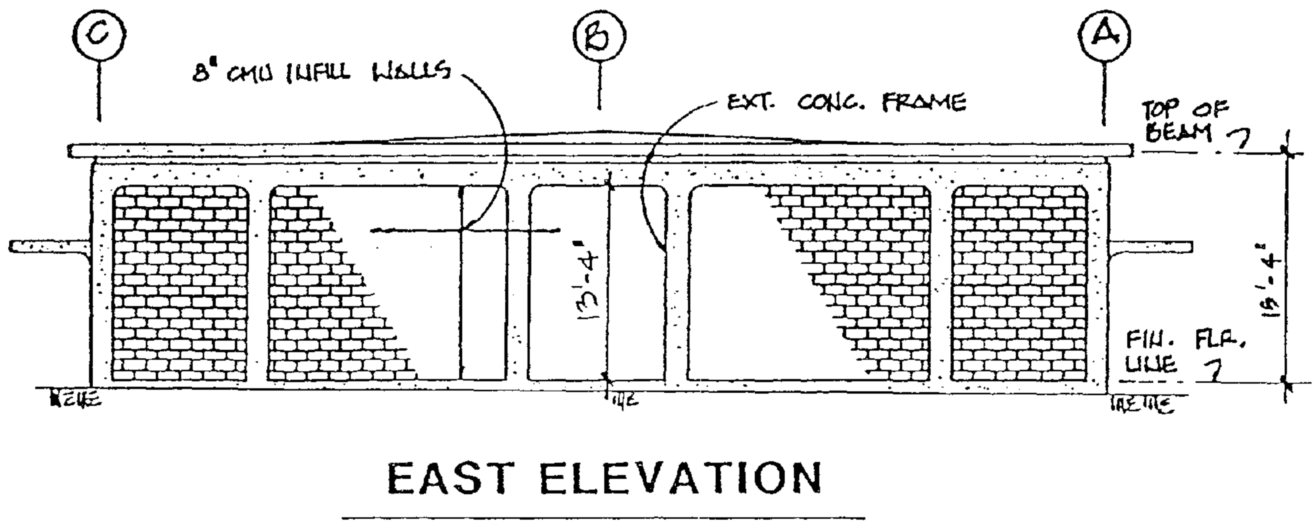
Exterior View, Example 1 Building.

FIGURE 12.2



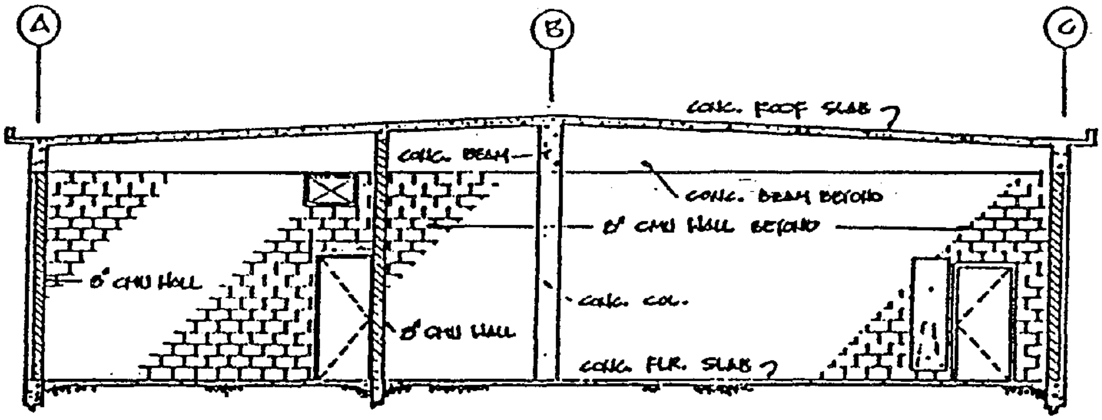
Floor Plan, Example 1 Building.

FIGURE 12.3



Exterior Elevation, Example 1 Building.

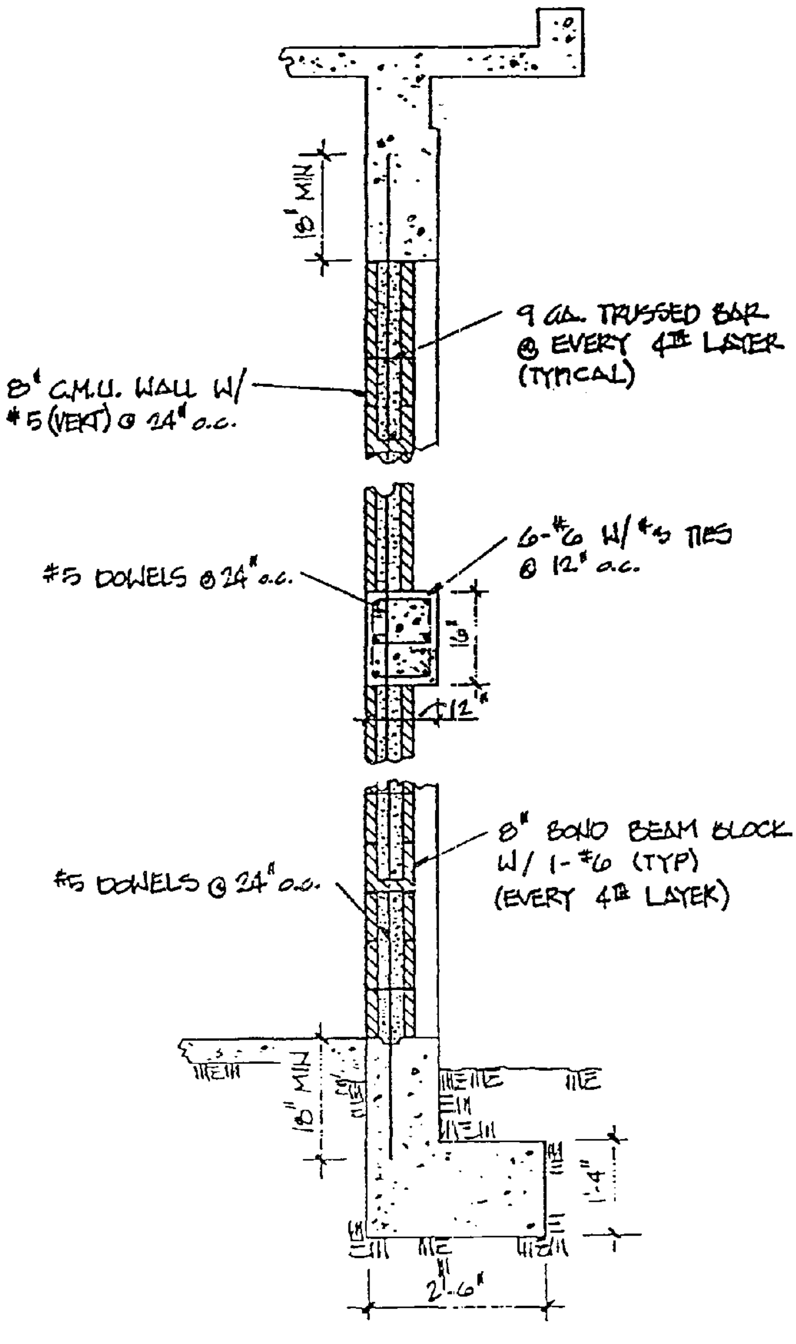
FIGURE 12.4



SECTION

Typical Transverse Section, Example 1 Building.

FIGURE 12.5



TYPICAL WALL SECTION

Typical Exterior Wall Section, Example 1 Building.

FIGURE 12.6

ATC-14 Field Data Sheet

Building Data

Year built: 1970 Year(s) remodelled? 1971 Date: 11-10-86
Area,sf: 7000 Length 102 Width 61 Photo Roll #: B-5
No. stories 1 Story height ______ Total ht. 13'4" To 15'4"

Construction Data

Gravity load structural system : CONCRETE FRAME
Exterior transverse walls: CONC. FRAME W/ MASONRY INFILL Openings? —
Exterior longitudinal walls: " " " " " Openings? 5%
Roof materials/framing : CONCRETE SLAB, BEAMS, AND GIRDERS
Intermediate floors/framing : N.A.
Ground floor : CONCRETE SLAB ON GRADE
Columns : NON-DUCTILE CONC - 14" SQUARE Foundation : SPREAD FOOTINGS AND GRADE BEAMS

Lateral Force Resisting System

	Longitudinal	Transverse
Diaphragms:	CONC. SLAB	SAME
Vertical Elements :	MASONRY WALLS INFILLED IN CONC. FRAME	SAME
Connections :		
Details :		

Building Classification: 7.3 & 9.1

ATC-14 Checklist

ATC structural checklist completed and attached? YES
ATC non-struc " " " " ? YES

General condition of structural? GOOD Evidence of settling? NO
Special features/comments : 20'x30' ADDITION TO BUILDING ON NORTH SIDE. ADDITION IS OF SIMILAR CONSTRUCTION.

Earthquake Damage Potential:

Limitted damage/ loss of function ( )
• no repairs required

Minor damage/ loss of function (✓)
• repairs while occupied

Major Damage/ loss of function ( )
• repairs required prior to occupation

Total Damage/ loss of function ( )
• demolition

ATC-14 Field Data Sheet, Example 1 Building.

FIGURE 12.7

Checklist 9. Concrete Frame Buildings With Infilled Walls of Unreinforced Masonry*

True/ False			Comments
True	RAPID EVALUATION OF SHEAR STRESS IN MASONRY INFILLED WALLS REQUIRED		
	MATERIALS		
True	7.3.6.1	Mortar quality—can't scrape with metal tool, and no large areas of eroded mortar.	
True	7.3.6.2	Diagonal cracks are less than 1.0 mm.	
True	7.3.6.3	No tranverse or diagonal cracks in concrete columns that encase infills wider than 1.0 mm.	
	STRUCTURAL ELEMENTS		
True	7.3.6.4	Concrete frames form a complete vertical system.	
True	7.3.6.5	No torsion.	
True	7.3.6.6	No vertical strength discontinuities.	
True	7.3.6.7	Infilled walls are continuous to the base of building.	
True	7.3.6.8	Infilled walls are continuous to the soffits of the frame beams.	
False	7.3.6.9	Height/thickness (H/t) of walls in one-story buildings are less than 14.	$H/t \approx 20$, but reinforced
NA	7.3.6.10	H/t of top story walls in multi-story buildings are less than 9.	
NA	7.3.6.11	H/t of walls in other stories of multi-story buildings are less than 20.	
NA	7.3.6.12	If L/D of wood diaphragms is greater than 3, there are nonstructural walls at less than 40-foot spacing.	
True	7.3.6.13	Infilled walls are not of cavity construction.	
True	7.3.6.14	Infilled panels are anchored to the concrete frames around the entire perimeter.	
NA	7.3.6.15	Chords around diaphragm openings greater than 50 percent of the width.	
	NONSTRUCTURAL ELEMENTS		
True	7.3.6.16	Cornices, parapets, and appendages are anchored.	

*See Chapter 7, Section 7.3.6, for detailed discussions on each of these checklist issues.

Structural Evaluation Checklist for Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry, Example 1 Building.

FIGURE 12.8

Checklist 12. **Reinforced Masonry Wall Buildings With Wood or Metal Deck Diaphragms***

True/ False			Comments
True		RAPID EVALUATION OF SHEAR STRESS IN MASONRY WALLS REQUIRED	
		MATERIALS	
True	9.1.6.1	No visible deterioration of masonry units.	
True	9.1.6.2	Mortar quality—can't scrape with metal tool, and no large areas of eroded mortar.	
		STRUCTURAL ELEMENTS	
True	9.1.6.3	Total vertical and horizontal wall reinforcing greater than .002 A_g, with .0007 A_g minimum in either direction. Maximum spacing of 48 inches. All vertical bars extend to the top of the wall.	
True	9.1.6.4	No torsion.	
True	9.1.6.5	No vertical strength discontinuities.	
True	9.1.6.6	No vertical mass or geometric irregularities.	
NA	9.1.6.7	Unblocked wood or untopped metal deck diaphragms have spans less than 40 feet and L/D less than or equal to 3 to 1.	
NA	9.1.6.8	No cross grain bending or tension in wood ledgers.	
NA	9.1.6.9	Masonry walls are attached to wood diaphragms with steel anchors or straps that are connected to a diaphragm cross tie.	
True	9.1.6.10	Wall anchors are spaced at 4 feet or less.	
True	9.1.6.11	Continuous cross ties between diaphragms chords.	
True	9.1.6.12	No openings adjacent to masonry walls larger than 8 ft.	
True	9.1.6.13	Diaphragm openings at walls are less than 25 percent of the length.	
True	9.1.6.14	All wall openings have trim reinforcing on all sides.	
		FOUNDATIONS	
True	9.1.6.15	All vertical reinforcing is doweled into the foundation.	
		NONSTRUCTURAL ELEMENTS	
NA	9.1.6.16	Veneer courses above the first floor are postively attached at less than 2 feet on center.	
True	9.1.6.17	Cornices, parapets, and appendages are anchored.	

*See Chapter 9, Section 9.1.6, for detailed discussions on each of these checklist issues.

Structural Evaluation Checklist for Reinforced Masonry Wall Buildings with Wood or Metal Deck Diaphragms, Example 1 Building.

FIGURE 12.9

Checklist 14. Nonstructural Elements

True/ False		Comments
	PARTITIONS	
False	11.2.1.1 All unreinforced masonry partitions are less than 8 feet tall. (LS)	
NA	11.2.1.2 Partitions are detailed to accommodate interstory drift.	
True	11.2.1.3 No partitions cross seismic joints.	
True	11.2.1.4 Partitions that only extend to ceiling are laterally braced at the top.	
	FURRING	
True	11.2.2 No structural elements are furred.	
	CEILINGS	
True	11.2.3.1 No suspended plaster or gyp board ceilings.	
True	11.2.3.2 No clips used to attach ceiling panels or tiles.	
False	11.2.3.3 No lay-in tiles are used.	
NA	11.2.3.4 No ceilings cross seismic joints.	
True	11.2.3.5 Ceilings are not required to provide lateral bracing of partitions.	
False	11.2.3.6 Ceiling edges are separated from structural walls.	
	LIGHT FIXTURES	
False	11.2.4.1 Multiple length flourescent fixtures are laterally braced or have secondary support.	
False	11.2.4.2 Flourescent light fixture lenses have safety chains or a positive attachment.	
False	11.2.4.3 Pendant fixtures can swing without contacting other elements.	
True	11.2.4.4 No double stem flourescent fixtures are used.	
	MECHANICAL EQUIPMENT	
True	11.2.5.1 Large equipment is positively attached to the structural system.	
False	11.2.5.2 Vibration isolated equipment has restraints to limit motion.	
True	11.2.5.3 No major equipment is suspended from the ceiling without seismic bracing. (LS)	

(Continued on next page)

Nonstructural Evaluation Checklist, Example 1 Building.

FIGURE 12.10

Checklist 14. (Continued)

True/ False			Comments
	PIPING		
NA	11.2.6.1	All pipes crossing seismic joints have flexible connectors.	
True	11.2.6.2	No pipes support other pipes.	
False	11.2.6.3	All pipe sleeve wall openings are at least 2 in. larger than the pipe.	
	DUCTS		
True	11.2.7.1	Long duct lines are laterally braced.	
True	11.2.7.2	No ducts are supported by pipes or other non-structural elements.	
NA	11.2.7.3	All ducts crossing seismic joints have flexible connections.	
	ELECTRICAL EQUIPMENT		
True	11.2.8.1	All electrical equipment is positively attached to the structure.	
NA	11.2.8.2	Rigid conduits or bus ducts have flexible sections at seismic joints.	
	ELEVATORS		
NA	11.2.9.1	Counterweights are well secured to the guiderails. (LS)	
NA	11.2.9.2	Motor/generator has restraints to limit deflections.	
NA	11.2.9.3	Control panels are bolted to the floor slabs.	
	CLADDING/GLAZING AND VENEER		
NA	11.2.10.1a	There are at least two bearing connections per panel. (LS)	
NA	11.2.10.1b	There are at least four connections for out-of-plane forces. (LS)	
NA	11.2.10.1c	Multi-story panels are attached at each level. (LS)	
NA	11.2.10.1d	Cladding elements can accommodate 3 in. of story drift. (LS)	
NA	11.2.10.1e	Veneer courses above the first story are positively attached at 2-ft maximum spacing. (LS)	

(Continued on next page)

FIGURE 12.10 (CONTINUED)

Checklist 14. (Continued)

True/ False			Comments
False	11.2.10.2a	Inserts used to attach wall elements are welded to or hooked around panel reinforcing steel. (LS)	
NA	11.2.10.2b	Glazing is isolated from in-plane motion. (LS)	
NA	11.2.10.2c	Connection eccentricities are considered. (LS)	
NA	11.2.10.2d	Connections with welded inserts appear to be capable of yielding before fracture. (LS)	
NA	11.2.10.2e	Connections were installed as prescribed in the drawings. (LS)	
NA	11.2.10.2f	Connections are not deteriorated or corroded. (LS)	
		PARAPETS, CORNICES, ORNAMENTATION, AND APPENDAGES	
NA	11.2.11.1	All unreinforced masonry parapets have H/t ratios less than 1.5. (LS)	
NA	11.2.11.2	All reinforced masonry parapets have H/t ratios less than 30. (LS)	
True	11.2.11.3	Concrete parapets with H/t ratios greater than 1.5 are reinforced. (LS)	
True	11.2.11.4	Appendages, cornices and other exterior wall ornamentations are anchored to the structure. (LS)	
		MEANS OF EGRESS	
True	11.2.12.1	Walls around stairs and corridors are not hollow tile or unreinforced masonry. (LS)	
True	11.2.12.2	All veneers, parapets, cornices, canopies, etc., above exits are well anchored. (LS)	
NA	11.2.12.3	Lay-in ceiling tiles are not used in exits or corridors. (LS)	

FIGURE 12.10 (CONTINUED)

RAPID EVALUATION OF SHEAR STRESSES IN MASONRY INFILL WALLS

See Sections 7.3.6.1 and 4.4.2

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

v_{AVG} = Average wall shear stress
V_j = Story shear in story j
A_w = Total area of walls in direction under consideration

Eqn (4.2) $$V_j = \frac{n+j}{n+1}\,\frac{W_j}{W}\,V$$

V = Base shear
W_j = Weight of story j
W = Total seismic dead load

Eqn (4.1) $$V = \frac{2.5A_a}{R_w}\,W$$

A_a = .40
R_w = 5 (for infilled wall buildings)

Therefore, $$V = \frac{2.5 \times .40}{5}\,W = \underline{\underline{.20\ W}}$$

Calculate Building Weights

Roof: (Average 5-inch concrete)
5/12 x 150 lb/ft^3 x 60.7 ft x 101.3 ft = 385 kips

Concrete Beams: Average depth = 3 ft, 12 in. wide, 11 total
11 x 3 ft x 1 ft x 60.7 ft x 150 lb/ft^3 = 300 kips

Concrete Girder: Average depth = 3 ft, 14 in. wide, L = 101.3 ft
3 ft x 14/12 x 101.3 ft x 150 lb./ft^3 = 53 kips

Interior Columns: 14 in. square, 4 total, use ½ height (H = 7.5 ft)
14/12 x 14/12 x 7.5 ft x 4 x 150 lb/ft^3 = 6 kips

Exterior Walls (including concrete columns): Grout every third cell in 8-in. CMU walls, use 60 psf of wall (see Amrhein, 1983, p. 294), H = 7.5 ft (½ height)
7.5 ft x (60.7 ft + 101.3 ft) x 2 x 60 psf = 145 kips

Rapid Evaluation of Shear Stresses in Masonry Infilled Walls, Example 1 Building.

FIGURE 12.11

Calculate Building Weights (Continued)

Interior Walls: Assume 200 ft of length
7.5 ft x 200 ft x 60 psf = 90 kips

Ceilings, Mechanical, Electrical, Plumbing: Assume 20 psf, total
20 psf x 60.7 ft x 101.3 ft = 123 kips

Therefore, Total Weight (W) = 1062 kips, and V = 0.20 W = 212 kips

Calculate Average Wall Shear Stress in Transverse Direction
Length of wall = 60.7 ft x 2 = 121.4 ft
Use equivalent solid thickness of 5.2 in (see Amrhein, 1983, p. 294)

A_W = 7575 in^2

Since V_j = V (Example 1 building is a one-story structure)

V_{AVG} = V/A_W = 212/7575 = 28 psi > 12.5 psi

Therefore, More Detailed Evaluation of Wall Stresses is Required

See Section 4.4.3

Calculate Average Wall Shear Stress in Longitudinal Direction
Length of wall = 101.3 ft x 2 = 202.6 ft

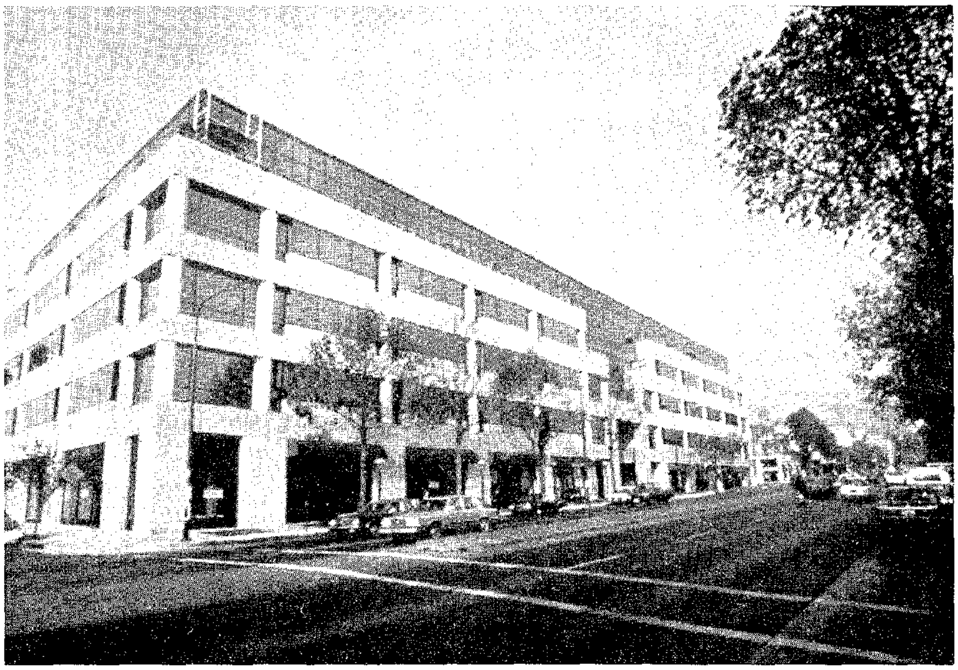
A_W = 202.6 ft x 12 in./ft x 56.2 in. = 12642 in^2

V_{AVG} = 212/12462 = 17 psi > 12.5 psi

Therefore, More Detailed Evaluation of Wall Stresses is Required

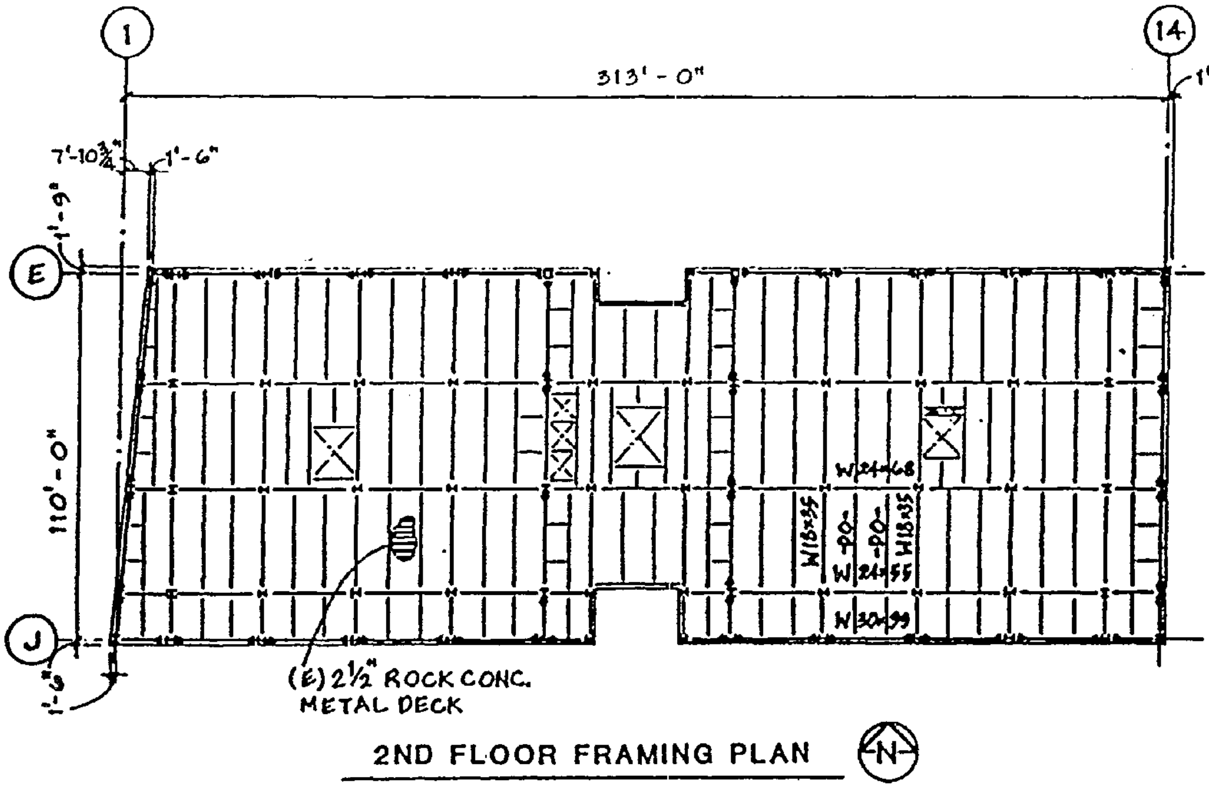
See Section 4.4.3

FIGURE 12.11 (CONTINUED)



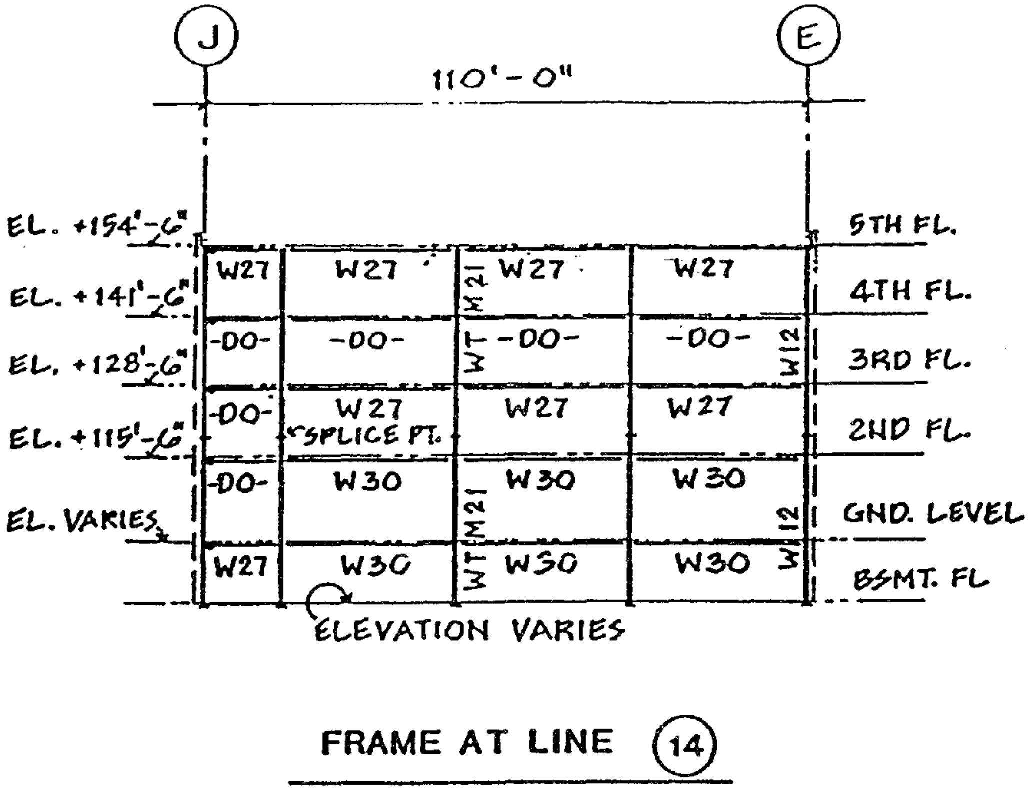
Exterior View, Example 2 Building.

FIGURE 12.12



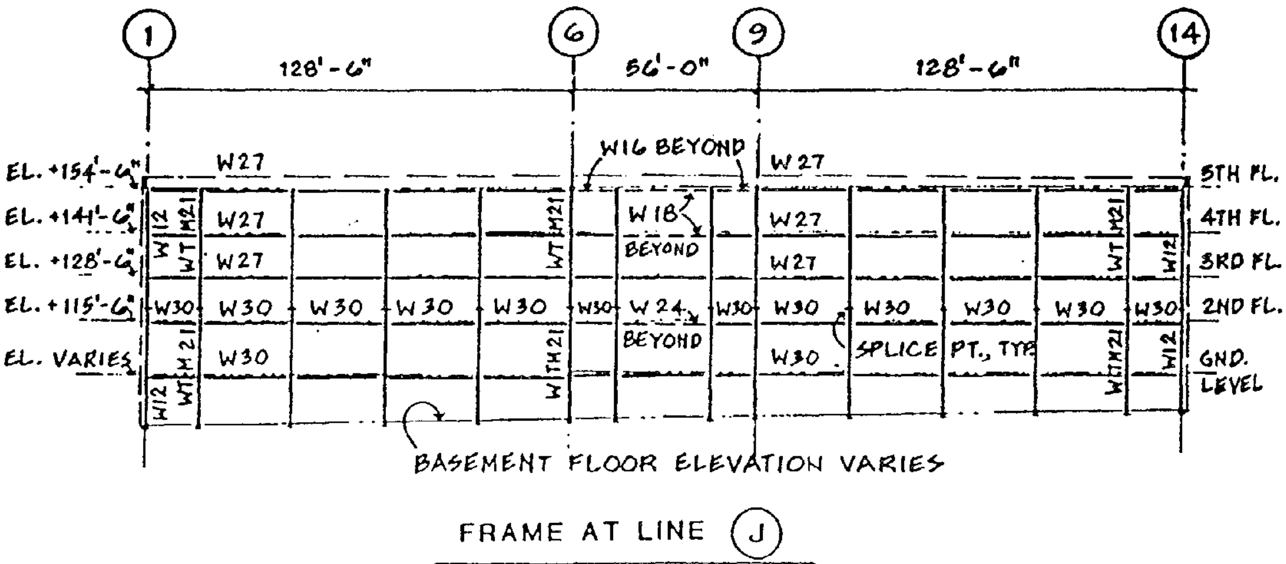
Typical Floor Plan, Example 2 Building.

FIGURE 12.13



Typical Transverse Moment Frame, Example 2 Building.

FIGURE 12.14



Typical Longitudinal Moment Frame, Example 2 Building.

FIGURE 12.15

ATC-14 Field Data Sheet

Building Data

Year built: 1986 Year(s) remodelled? ______ Date: 1-30-87
Area,sf: 35,700 Length 316' Width 113' Photo Roll #: ______
No. stories 5 Story height 13' Total ht. 68'

Construction Data

Gravity load structural system : Steel frame
Exterior transverse walls: Concrete & glass panels Openings? 70%
Exterior longitudinal walls: " " " Openings? 70%
Roof materials/framing : Concrete slab with steel framing
Intermediate floors/framing : " " " " "
Ground floor : Concrete slab on steel frames
Columns : steel Foundation : spread footings

Lateral Force Resisting System

	Longitudinal	Transverse
Diaphragms:	reinforced concrete slabs	Same
Vertical Elements :	Steel Frames	
Connections :	moment resisting	
Details :	ductile	

Building Classification: 6.1 - Steel moment frame

ATC-14 Checklist

ATC structural checklist completed and attached? yes
ATC non-struc " " " " ? yes

General condition of structural? Good Evidence of settling? no
Special features/comments : one story of parking below ground level

Earthquake Damage Potential:

Limitted damage/ loss of function ( )
• no repairs required

Minor damage/ loss of function (✓)
• repairs while occupied

Major Damage/ loss of function ( )
• repairs required prior to occupation

Total Damage/ loss of function ( )
• demolition

ATC-14 Field Data Sheet, Example 2 Building.

FIGURE 12.16

Checklist 2. Steel Moment Frame Buildings*

True/ False			Comments
True	RAPID EVALUATION OF DRIFT REQUIRED		
	MATERIALS		
True	6.1.6.1	No significant rusting, corrosion, or other deterioration of steel.	
	STRUCTURAL ELEMENTS		
True	6.1.6.2	No masonry infills. See Section 6.5	
True	6.1.6.3	Metal deck has topping slab.	
True	6.1.6.4	No torsion.	
True	6.1.6.5	No vertical strength discontinuities.	
True	6.1.6.6	No vertical mass or geometric irregularities.	
True	6.1.6.7	Compact sections in moment frames.	
True	6.1.6.8	No pounding of adjacent structures.	
True	6.1.6.9	Full penetration welds at moment connections.	
True	6.1.6.10	Good column splice details—flanges and web.	
True	6.1.6.11	Good shear transfer mechanism between diaphragms and frames.	
True	6.1.6.12	Chords around diaphragm openings greater than 50 percent of the width.	
NA	6.1.6.13	Large tensile capacity at re-entrant corners or other plan irregularities.	
True	6.1.6.14	Web penetrations are less than d/4 and are located in center half of beams.	
True	6.1.6.15	Web thicknesses within joints meet AISC criteria.	
	FOUNDATIONS		
True	6.1.6.16	Columns are well anchored to foundation.	
	NONSTRUCTURAL ELEMENTS		
True	6.1.6.17	Cornices, parapets, and appendages are anchored.	
True	6.1.6.18	Exterior cladding and veneer are well anchored.	

*See Chapter 6, Section 6.1.6, for detailed discussions on each of these checklist issues.

Structural Evaluation Checklist for Steel Moment Frame Buildings, Example 2 Building.

FIGURE 12.17

Checklist 14. Nonstructural Elements

True/ False		Comments
	PARTITIONS	
True	11.2.1.1 All unreinforced masonry partitions are less than 8 feet tall. (LS)	
False	11.2.1.2 Partitions are detailed to accommodate interstory drift.	
NA	11.2.1.3 No partitions cross seismic joints.	
True	11.2.1.4 Partitions that only extend to ceiling are laterally braced at the top.	
	FURRING	
True	11.2.2 No structural elements are furred.	
	CEILINGS	
True	11.2.3.1 No suspended plaster or gyp board ceilings.	
True	11.2.3.2 No clips used to attach ceiling panels or tiles.	
False	11.2.3.3 No lay-in tiles are used.	
NA	11.2.3.4 No ceilings cross seismic joints.	
True	11.2.3.5 Ceilings are not required to provide lateral bracing of partitions.	
True	11.2.3.6 Ceiling edges are separated from structural walls.	
	LIGHT FIXTURES	
False	11.2.4.1 Multiple length flourescent fixtures are laterally braced or have secondary support.	
False	11.2.4.2 Flourescent light fixture lenses have safety chains or a positive attachment.	
False	11.2.4.3 Pendant fixtures can swing without contacting other elements.	
True	11.2.4.4 No double stem flourescent fixtures are used.	
	MECHANICAL EQUIPMENT	
True	11.2.5.1 Large equipment is positively attached to the structural system.	
True	11.2.5.2 Vibration isolated equipment has restraints to limit motion.	
True	11.2.5.3 No major equipment is suspended from the ceiling without seismic bracing. (LS)	

(Continued on next page)

Nonstructural Evaluation Checklist, Example 2 Building.

FIGURE 12.18

Checklist 14. (Continued)

True/ False			Comments
	PIPING		
NA	11.2.6.1	All pipes crossing seismic joints have flexible connectors.	
True	11.2.6.2	No pipes support other pipes.	
True	11.2.6.3	All pipe sleeve wall openings are at least 2 in. larger than the pipe.	
	DUCTS		
False	11.2.7.1	Long duct lines are laterally braced.	
True	11.2.7.2	No ducts are supported by pipes or other non-structural elements.	
NA	11.2.7.3	All ducts crossing seismic joints have flexible connections.	
	ELECTRICAL EQUIPMENT		
True	11.2.8.1	All electrical equipment is positively attached to the structure.	
NA	11.2.8.2	Rigid conduits or bus ducts have flexible sections at seismic joints.	
	ELEVATORS		
True	11.2.9.1	Counterweights are well secured to the guiderails. (LS)	
True	11.2.9.2	Motor/generator has restraints to limit deflections.	
True	11.2.9.3	Control panels are bolted to the floor slabs.	
	CLADDING/GLAZING AND VENEER		
True	11.2.10.1a	There are at least two bearing connections per panel. (LS)	
True	11.2.10.1b	There are at least four connections for out-of-plane forces. (LS)	
True	11.2.10.1c	Multi-story panels are attached at each level. (LS)	
True	11.2.10.1d	Cladding elements can accommodate 3 in. of story drift. (LS)	
True	11.2.10.1e	Veneer courses above the first story are positively attached at 2-ft maximum spacing. (LS)	

(Continued on next page)

FIGURE 12.18 (CONTINUED)

Checklist 14. (Continued)

True/ False			Comments
True	11.2.10.2a	Inserts used to attach wall elements are welded to or hooked around panel reinforcing steel. (LS)	
True	11.2.10.2b	Glazing is isolated from in-plane motion. (LS)	
True	11.2.10.2c	Connection eccentricities are considered. (LS)	
True	11.2.10.2d	Connections with welded inserts appear to be capable of yielding before fracture. (LS)	
True	11.2.10.2e	Connections were installed as prescribed in the drawings. (LS)	
True	11.2.10.2f	Connections are not deteriorated or corroded. (LS)	
	PARAPETS, CORNICES, ORNAMENTATION, AND APPENDAGES		
NA	11.2.11.1	All unreinforced masonry parapets have H/t ratios less than 1.5. (LS)	
NA	11.2.11.2	All reinforced masonry parapets have H/t ratios less than 30. (LS)	
NA	11.2.11.3	Concrete parapets with H/t ratios greater than 1.5 are reinforced. (LS)	
True	11.2.11.4	Appendages, cornices and other exterior wall ornamentations are anchored to the structure. (LS)	
	MEANS OF EGRESS		
True	11.2.12.1	Walls around stairs and corridors are not hollow tile or unreinforced masonry. (LS)	
True	11.2.12.2	All veneers, parapets, cornices, canopies, etc., above exits are well anchored. (LS)	
False	11.2.12.3	Lay-in ceiling tiles are not used in exits or corridors. (LS)	

FIGURE 12.18 (CONTINUED)

RAPID EVALUATION OF DRIFT

Step 1: Dead Load (stories 1 thru 4): 75 psf (slab and beams)
15 psf (partitions and msc)
90 psf

Area (stories 1 thru 4): 113 ft x 316 ft = 35708 ft^2

$W_1 = W_2 = W_3 = W_4 = 90$ psf x 35708 ft^2 x 1 kip/1000 lb. = **3214 kips**

Step 2: Dead Load (Roof): 75 psf (slab and beams)
15 psf (partitions and msc)
90 psf

Area = 108 ft x 304 ft = 32832 ft^2

DL (penthouse) = 75 psf, Area = 32 ft x 190 ft = 6080 ft^2

W_{roof} = (90 psf (32832 ft^2) + 75 psf (6080 ft^2)) 1kip/1000 lb = 3411 kips

Step 3: Base Shear (Equation 4.1)

A_a = .40 (Fig. 3.1)
R_w = 12 (Table 4.4)
W = 4 (3214 kips) + 3411 kips = 16,267 kips

$$V = \frac{2.5A_a}{R_w}W$$

$$V = \frac{2.5\ (0.4)}{12}(16{,}267) = \underline{\underline{1356\ \text{kips}}}$$

Step 4: Story Shears (Equation 4.2)

n = Number of stories = 5
W_j = Total weight of all stories above
W = 16267 kips
V = 1356 kips

$$V_j = \frac{n+j}{n+1}\ \frac{W_j}{W}\ V$$

a. First Story

$$V_1 = \frac{5+1}{5+1}\ \frac{16{,}267\ \text{kips}}{16{,}267\ \text{kips}}\ (1356\ \text{kips}) = \underline{\underline{1356\ \text{kips}}}$$

Rapid Estimation of Story Drift, Example 2 Building.

FIGURE 12.19

b. Second Story

$$V_2 = \frac{5+2}{5+1}\ \frac{13{,}053\ \text{kips}}{16{,}267\ \text{kips}}\ (1356\ \text{kips}) = \underline{\underline{1269\ \text{kips}}}$$

c. Third Story

$$V_3 = \frac{5+3}{5+1}\ \frac{9{,}839\ \text{kips}}{16{,}267\ \text{kips}}\ (1356\ \text{kips}) = \underline{\underline{1093\ \text{kips}}}$$

d. Fourth Story

$$V_4 = \frac{5+4}{5+1}\ \frac{6{,}625\ \text{kips}}{16{,}267\ \text{kips}}\ (1356\ \text{kips}) = \underline{\underline{828\ \text{kips}}}$$

e. Fifth Story

$$V_5 = \frac{5+5}{5+1}\ \frac{3{,}411\ \text{kips}}{16{,}267\ \text{kips}}\ (1356\ \text{kips}) = \underline{\underline{474\ \text{kips}}}$$

Step 5: Building Model

2nd Floor thru 5th Floor Plan

Roof Plan

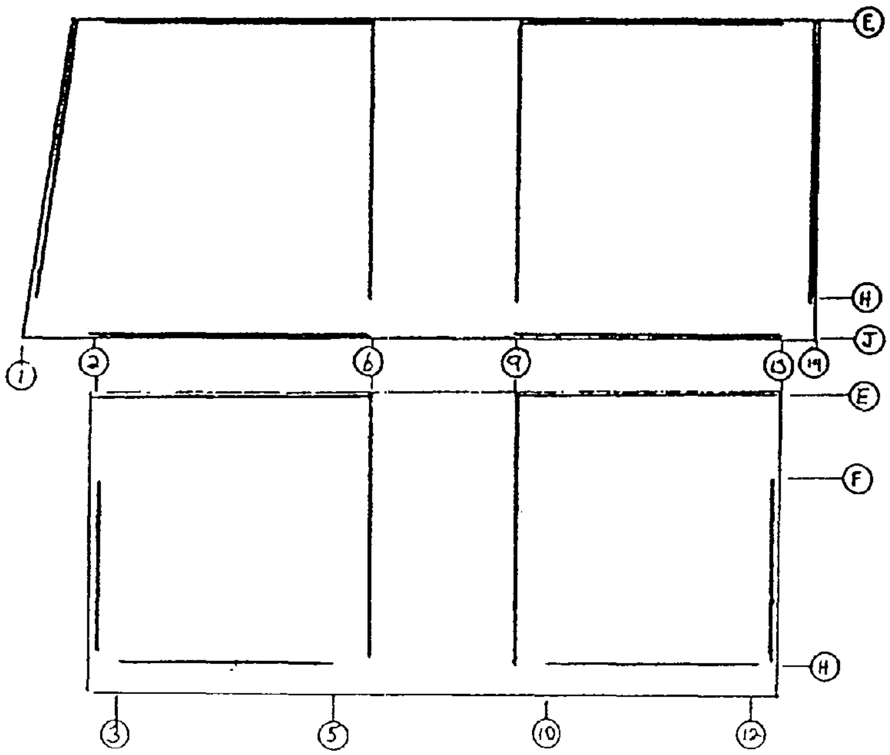
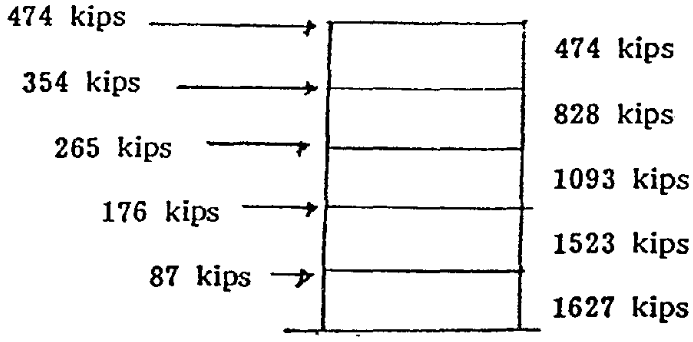


FIGURE 12.19 (CONTINUED)

Step 5 (Continued):



Step 6: Drift Evaluation

$$\Delta = \frac{k_b + k_c}{k_b k_c} \frac{h}{348{,}000} V_c$$

$k_b = (I/L)_{beam}$ h = Story height (in.)

$k_c = (I/L)_{column}$ V_c = Average shear in each column

a. First Story (North-South direction)

Columns (16)
Size: WTM21 x 182
$I = 4730 \text{ in.}^4$
$L = 16 \text{ ft } (12 \text{ in./ft}) = 192 \text{ in.}$

Beams
Size: W30 x 99
$I = 3990 \text{ in.}^4$
$L = 384 \text{ in.}$

$V_c = 1356 \text{ kips}/16 \text{ col.} = 85 \text{ kips/column}$

$k_b = (I/L)_{beam} = 3990 \text{ in.}^4/384 \text{ in.} = 10.4 \text{ in.}^3$

$k_c = (I/L)_{column} = 4730 \text{ in.}^4/192 \text{ in.} = 24.6 \text{ in.}^3$

$$\Delta = \frac{(10.4 \text{ in.}^3 + 24.6 \text{ in.}^3)}{(10.4 \text{ in.}^3)(24.6 \text{ in.}^3)} \frac{(192 \text{ in.})}{348{,}000} (85 \text{ kips/column}) = .0064 > .005$$

Therefore, use full frame analysis with UBC lateral loads

FIGURE 12.19 (CONTINUED)

DRIFT DETERMINED FROM LATERAL FORCE PROCEDURE

UBC Lateral Analysis (ICBO, 1985)

$$V = \frac{A_v I C}{R_w} W$$

$W = 16{,}267$ kips
$A_v = .4$
$I = 1$
$R_w = 12$
$C = .8S/T^{2/3}$ $\quad S = 1.5,\ T = C_t(h_n)^{.75} = .035(68)^{.75} = .83$ secs
$\quad = .8 \times 1.5/.83 = 1.45$

$$V = \frac{.4 \times 1.0 \times 1.45}{12} W = .048\ W = 784 \text{ kips}$$

Story	w (kips)	h (feet)	wh
1	3214	16	51,424
2	3214	29	93,206
3	3214	42	134,988
4	3214	55	176,770
5	3214	68	231,948
Σ			688,336

$$F_x = \frac{V(w_x h_x)}{\Sigma wh}$$

$$F_1 = \frac{784 \text{ kips } (51{,}424)}{688{,}336} = 59 \text{ kips}$$

$F_2 = 106$ kips

$F_3 = 154$ kips

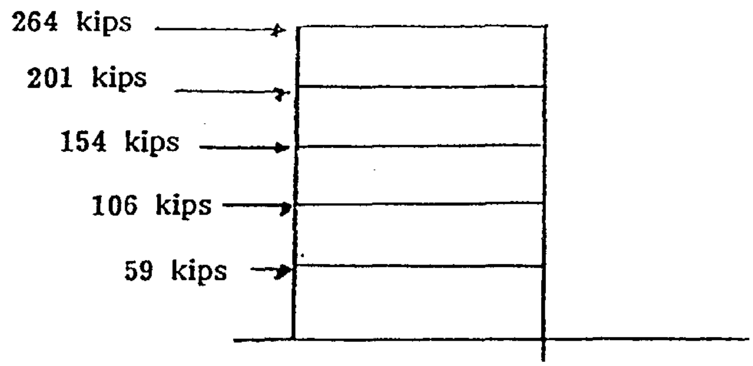
$F_4 = 201$ kips

$F_5 = 264$ kips

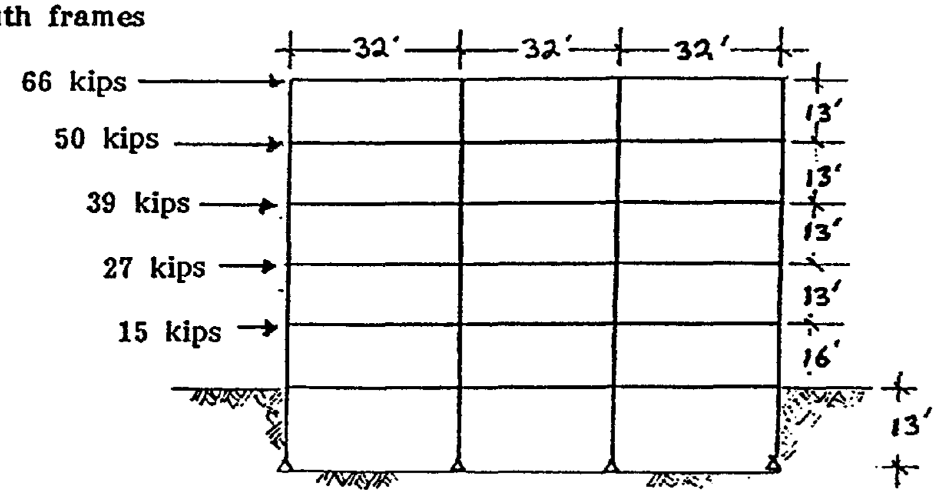
Drift Determined From Equivalent Lateral Force Procedure, Example 2 Building.

FIGURE 12.20

DRIFT DETERMINED FROM LATERAL FORCE PROCEDURE



For computer analysis of North-South frame on Line 6 or 9, use 1/4 loads since there are 4 North-South frames



<u>First Story Drift:</u>

$$\theta = \frac{\Delta}{h} = \frac{.58 \text{ in.}}{16 \text{ ft } (12 \text{ in./ft})} = .003 < .005$$

Check, $\theta_{max} = .04/R_W = .0033 > .003$ (SEAOC, 1985)

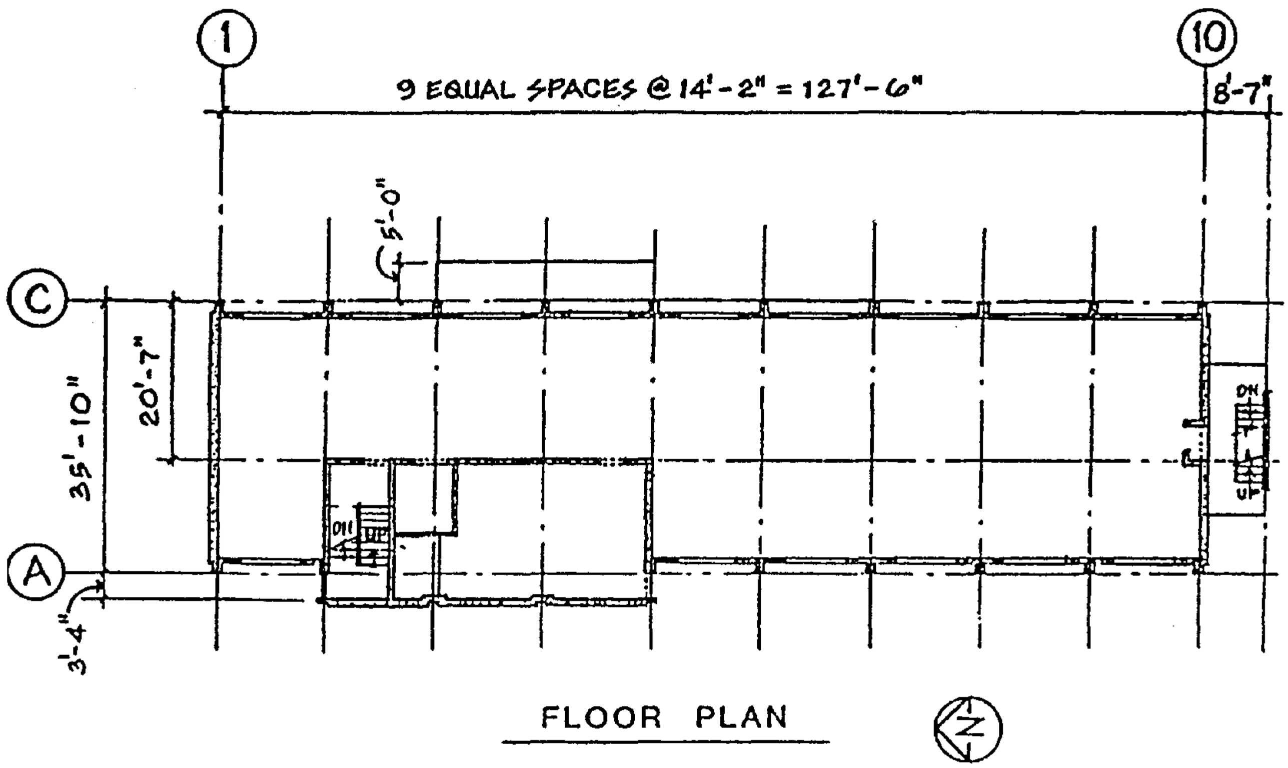
Therefore, <u>the drift is OK for the ATC-14 force level for existing buildings</u>

(Note: Elastic frame analysis used to calculate these drifts is not included)

FIGURE 12.20 (CONTINUED)

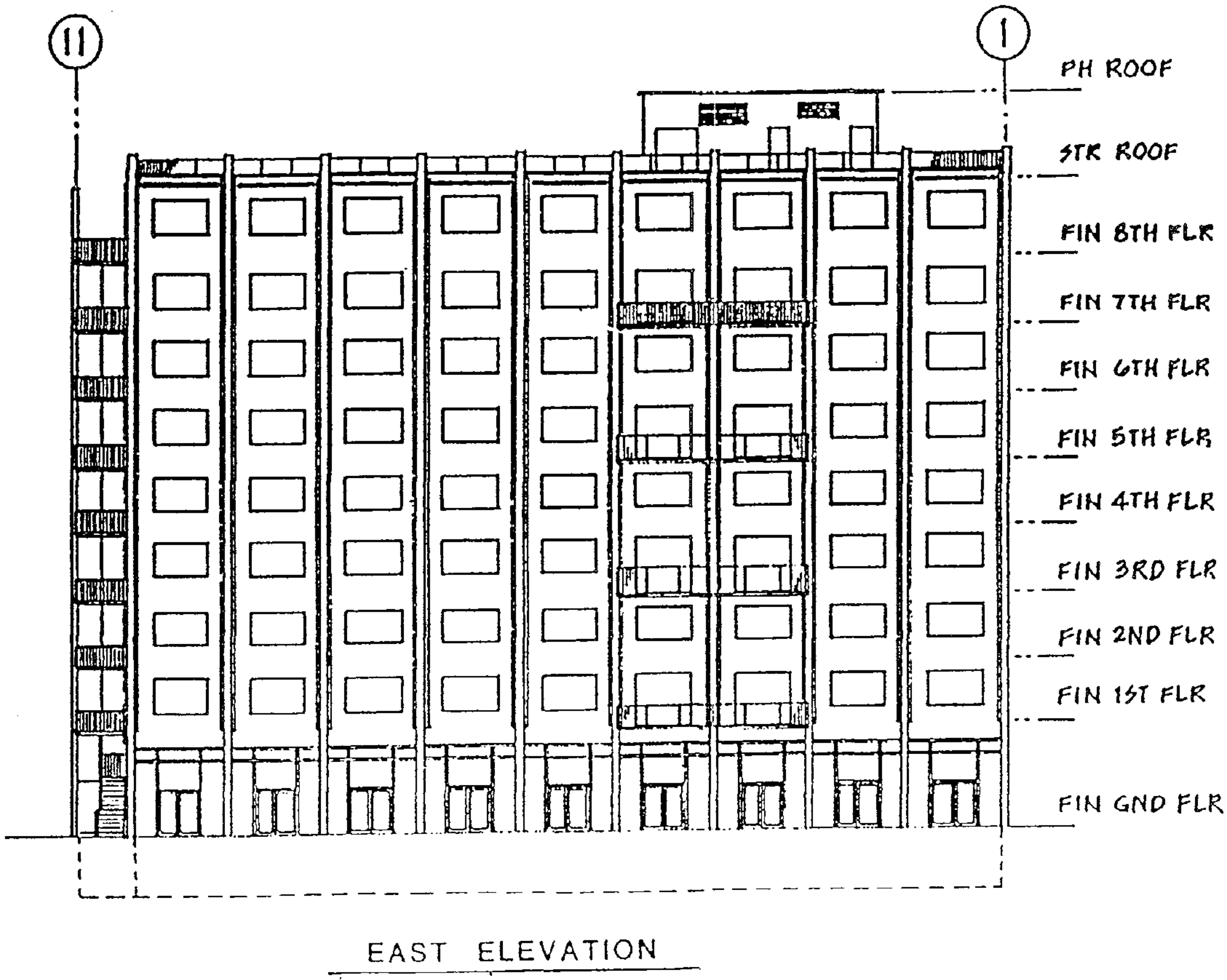
Exterior View, Example 3 Building.

FIGURE 12.21



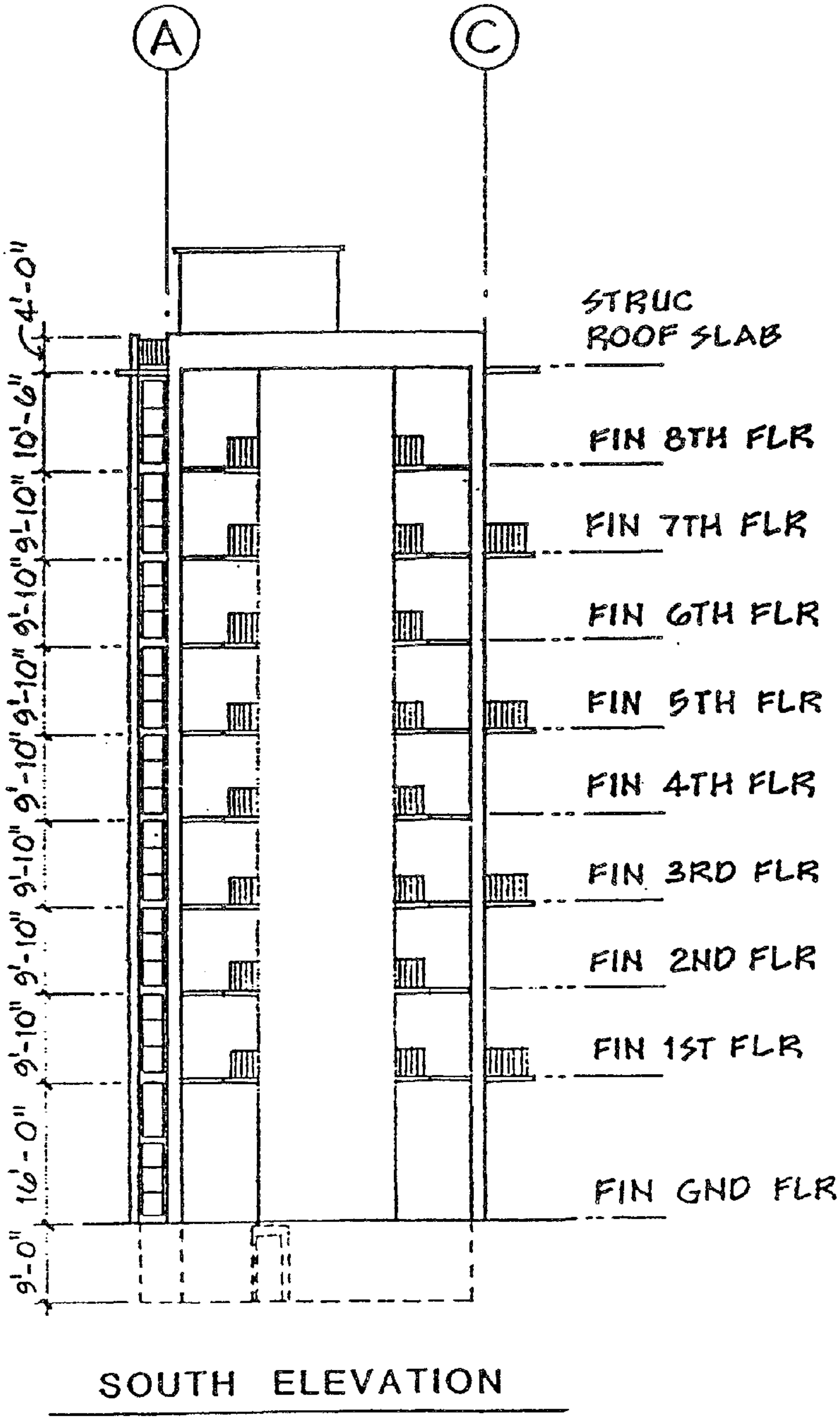
Typical Floor Plan, Example 3 Building.

FIGURE 12.22



Longitudinal Elevation, Example 3 Building.

FIGURE 12.23



Transverse Elevation, Example 3 Building.

FIGURE 12.24

ATC-14 Field Data Sheet

Building Data

Year built: 1961 Year(s) remodelled? — Date: 8/10/86
Area,sf: 41000 Length 128 Width 36' Photo Roll #: -
No. stories 9 Story height 9'10" TYP 16' - 1st FLOOR Total ht. 95'

Construction Data

Gravity load structural system : ONE WAY JOISTS TO BEAMS TO EXT. COLS OR WALLS
Exterior transverse walls: CONCRETE SHEAR WALLS Openings? 0% OR DOOR
Exterior longitudinal walls: CONCRETE FRAME/WALL Openings? 40%
Roof materials/framing : ONE WAY JOIST & BEAMS AND FLAT SLAB
Intermediate floors/framing : SAME AS FLOORS
Ground floor : CONC. SLAB ON GRADE - 4"
Columns : 16" x 24" CONC. Foundation : 24" AND 30" Ø CAISSONS

Lateral Force Resisting System

	Longitudinal	Transverse
Diaphragms:	CAST-IN-PLACE SLABS	SAME
Vertical Elements :	CONC. FRAME WALL AND CONC. WALL	CONC. SHEAR WALLS
Connections :	CAST-IN-PLACE	
Details :	WIDELY SPACED COL. TIES	ALL LONG. REBAR NOT DOWELLED INTO CAISSONS

Building Classification: 7.2 - CONC. SHEAR WALLS

ATC-14 Checklist

ATC structural checklist completed and attached? YES
ATC non-struc " " " " ? YES

General condition of structural? FAIR Evidence of settling? SLIGHT
Special features/comments : EXPANSION JOINTS PROVIDED AT EXT. LONG. FRAMES. SOFT STORY AT FIRST FLOOR LONGITUDINAL DUE TO LOSS OF COL. STIFFNESS FOR LOUNGE AREA. TORSION COULD BE A PROBLEM

Earthquake Damage Potential:

Limitted damage/ loss of function ( )
• no repairs required

Minor damage/ loss of function ( )
• repairs while occupied

Major Damage/ loss of function (✓)
• repairs required prior to occupation

Total Damage/ loss of function ( )
• demolition

ATC-14 Field Data Sheet, Example 3 Building.

FIGURE 12.25

Checklist 8. Concrete Shear Wall Buildings*

True/ False			Comments
True		RAPID EVALUATION OF SHEAR STRESS IN CONCRETE WALLS REQUIRED	
		MATERIALS	
True	7.2.6.1	Diagonal wall cracks are less than 1.0 mm.	
True	7.2.6.2	No significant deterioration of rebar.	Some rusted and spalled rebar to be repaired
NA	7.2.6.3	No evidence of spalling at post-tensioning or end fittings.	
		STRUCTURAL ELEMENTS	
True	7.2.6.4	Wall reinforcing greater than 0.0025 A_g each way, with maximum spacing of 18 inches.	
True	7.2.6.5	Metal deck has topping slab.	
False	7.2.6.6	No torsion.	Coupled shear wall on line 10 is not as stiff as the solid wall on line 1. Also have longitudinal eccentricity due to the stiffness of wall on line A at bathrooms
False	7.2.6.7	No vertical strength discontinuities.	Tall first story and loss of frame stiffness in longitudinal direction.
True	7.2.6.8	No vertical mass or geometric irregularities.	
True	7.2.6.9	Walls are continuous to the foundation.	
True	7.2.6.10	Reinforcing in each diaphragm to transfer loads to walls.	
False	7.2.6.11	Wall H/D ratios are less than 4.	Various walls have H/D > 4 especially the transverse walls at the bathroom
True	7.2.6.12	Chords around diaphragm openings greater than 50 percent of the width.	
True	7.2.6.13	Large tensile capacity at re-entrant corners or other plan irregularities.	
False	7.2.6.14	Diaphragm openings at walls are less than 25 percent of the length.	Openings around stair walls
False	7.2.6.15	For walls with H/D greater than 2.0, boundary elements have spirals or ties at spacing less than 8 d_b.	None of the shear walls have ductile details for the boundary elements.
	7.2.6.16	Special reinforcement around all wall openings.	
False	7.2.6.17	Stirrups in coupling beams are spaced at 8 d_b and are well anchored into core.	Wall on line 10
		FOUNDATIONS	
True	7.2.6.18	All vertical wall reinforcing is doweled into the foundation.	
		NONSTRUCTURAL ELEMENTS	
True	7.2.6.19	Cornices, parapets, and appendages are anchored.	
NA	7.2.6.20	Exterior cladding and veneer are well anchored.	

*See Chapter 7, Section 7.2.6, for detailed discussions on each of these checklist issues.

Structural Evaluation Checklist for Concrete Shear Wall Buildings, Example 3 Building.

FIGURE 12.26

Checklist 14. Nonstructural Elements

True/False			Comments
	PARTITIONS		
NA	11.2.1.1	All unreinforced masonry partitions are less than 8 feet tall. (LS)	
False	11.2.1.2	Partitions are detailed to accommodate interstory drift.	
NA	11.2.1.3	No partitions cross seismic joints.	
False	11.2.1.4	Partitions that only extend to ceiling are laterally braced at the top.	
	FURRING		
True	11.2.2	No structural elements are furred.	
	CEILINGS		
True	11.2.3.1	No suspended plaster or gyp board ceilings.	
True	11.2.3.2	No clips used to attach ceiling panels or tiles.	
False	11.2.3.3	No lay-in tiles are used.	
True	11.2.3.4	No ceilings cross seismic joints.	
False	11.2.3.5	Ceilings are not required to provide lateral bracing of partitions.	
False	11.2.3.6	Ceiling edges are separated from structural walls.	
	LIGHT FIXTURES		
NA	11.2.4.1	Multiple length flourescent fixtures are laterally braced or have secondary support.	
False	11.2.4.2	Flourescent light fixture lenses have safety chains or a positive attachment.	
NA	11.2.4.3	Pendant fixtures can swing without contacting other elements.	
True	11.2.4.4	No double stem flourescent fixtures are used.	
	MECHANICAL EQUIPMENT		
True	11.2.5.1	Large equipment is positively attached to the structural system.	
True	11.2.5.2	Vibration isolated equipment has restraints to limit motion.	
True	11.2.5.3	No major equipment is suspended from the ceiling without seismic bracing. (LS)	

(Continued on next page)

Nonstructural Evaluation Checklist, Example 3 Building.

FIGURE 12.27

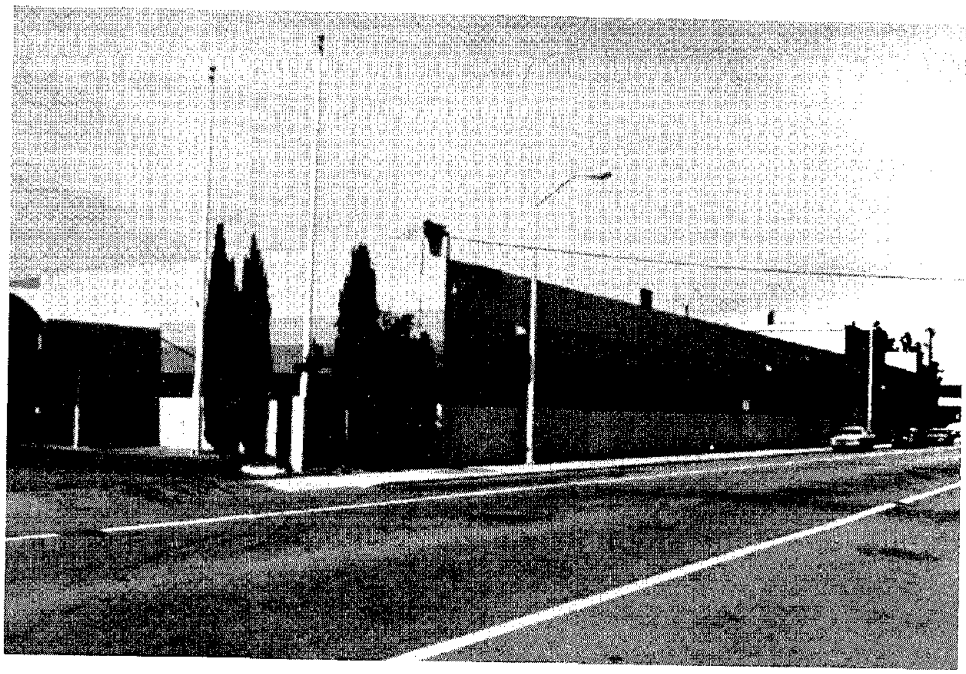
True/ False			Comments
	PIPING		
NA	11.2.6.1	All pipes crossing seismic joints have flexible connectors.	
True	11.2.6.2	No pipes support other pipes.	
False	11.2.6.3	All pipe sleeve wall openings are at least 2 in. larger than the pipe.	
	DUCTS		
False	11.2.7.1	Long duct lines are laterally braced.	
True	11.2.7.2	No ducts are supported by pipes or other non-structural elements.	
NA	11.2.7.3	All ducts crossing seismic joints have flexible connections.	
	ELECTRICAL EQUIPMENT		
True	11.2.8.1	All electrical equipment is positively attached to the structure.	
NA	11.2.8.2	Rigid conduits or bus ducts have flexible sections at seismic joints.	
	ELEVATORS		
True	11.2.9.1	Counterweights are well secured to the guiderails. (LS)	
False	11.2.9.2	Motor/generator has restraints to limit deflections.	
True	11.2.9.3	Control panels are bolted to the floor slabs.	
	CLADDING/GLAZING AND VENEER		
NA	11.2.10.1a	There are at least two bearing connections per panel. (LS)	
NA	11.2.10.1b	There are at least four connections for out-of-plane forces. (LS)	
NA	11.2.10.1c	Multi-story panels are attached at each level. (LS)	
NA	11.2.10.1d	Cladding elements can accommodate 3 in. of story drift. (LS)	
NA	11.2.10.1e	Veneer courses above the first story are positively attached at 2-ft maximum spacing. (LS)	

(Continued on next page)

FIGURE 12.27 (CONTINUED)

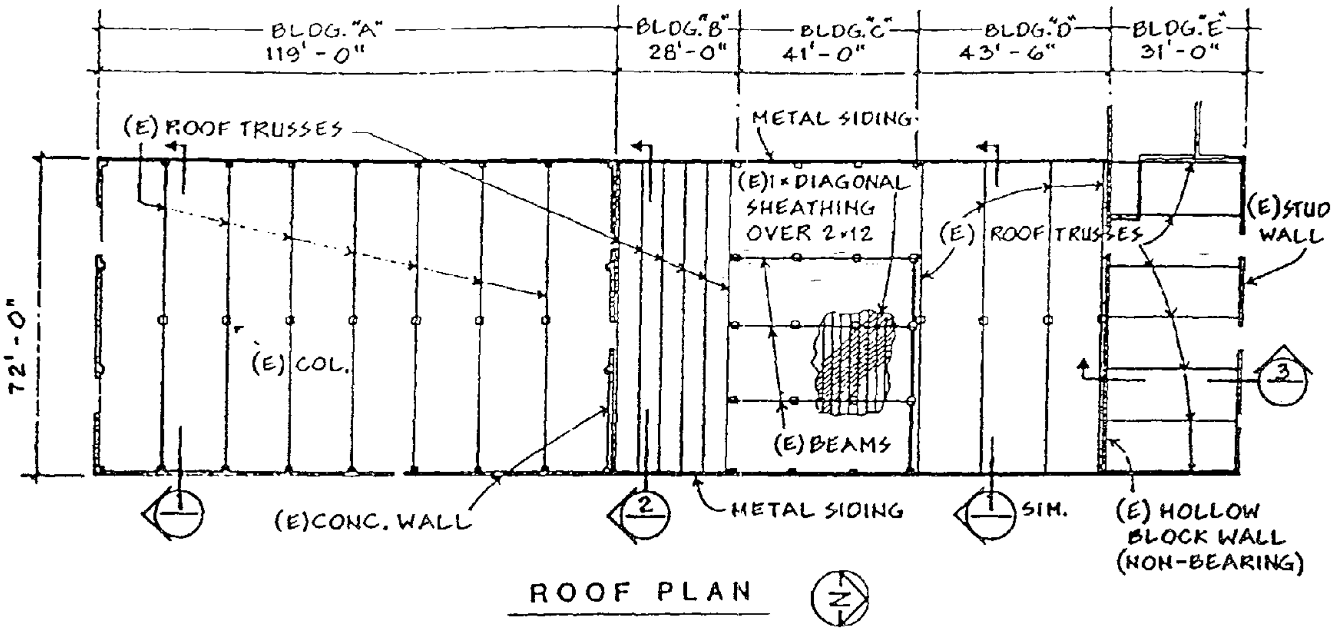
True/ False			Comments
NA	11.2.10.2a	Inserts used to attach wall elements are welded to or hooked around panel reinforcing steel. (LS)	
NA	11.2.10.2b	Glazing is isolated from in-plane motion. (LS)	
NA	11.2.10.2c	Connection eccentricities are considered. (LS)	
NA	11.2.10.2d	Connections with welded inserts appear to be capable of yielding before fracture. (LS)	
NA	11.2.10.2e	Connections were installed as prescribed in the drawings. (LS)	
NA	11.2.10.2f	Connections are not deteriorated or corroded. (LS)	
		PARAPETS, CORNICES, ORNAMENTATION, AND APPENDAGES	
NA	11.2.11.1	All unreinforced masonry parapets have H/t ratios less than 1.5. (LS)	
NA	11.2.11.2	All reinforced masonry parapets have H/t ratios less than 30. (LS)	
True	11.2.11.3	Concrete parapets with H/t ratios greater than 1.5 are reinforced. (LS)	
NA	11.2.11.4	Appendages, cornices and other exterior wall ornamentations are anchored to the structure. (LS)	
		MEANS OF EGRESS	
True	11.2.12.1	Walls around stairs and corridors are not hollow tile or unreinforced masonry. (LS)	
NA	11.2.12.2	All veneers, parapets, cornices, canopies, etc., above exits are well anchored. (LS)	
False	11.2.12.3	Lay-in ceiling tiles are not used in exits or corridors. (LS)	

FIGURE 12.27 (CONTINUED)



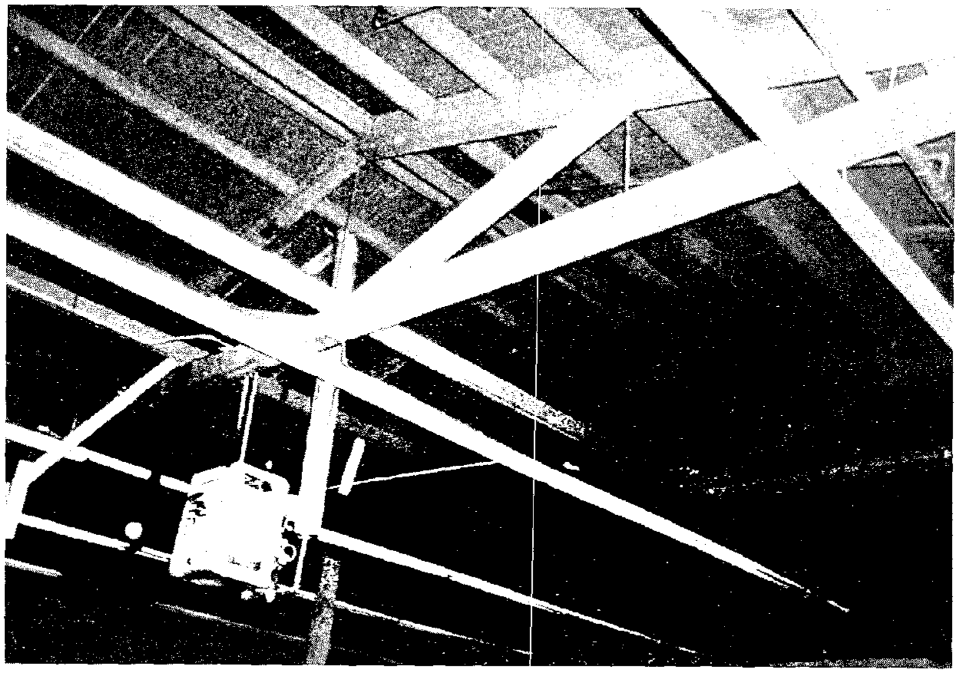
Exterior View, Example 4 Building.

FIGURE 12.28



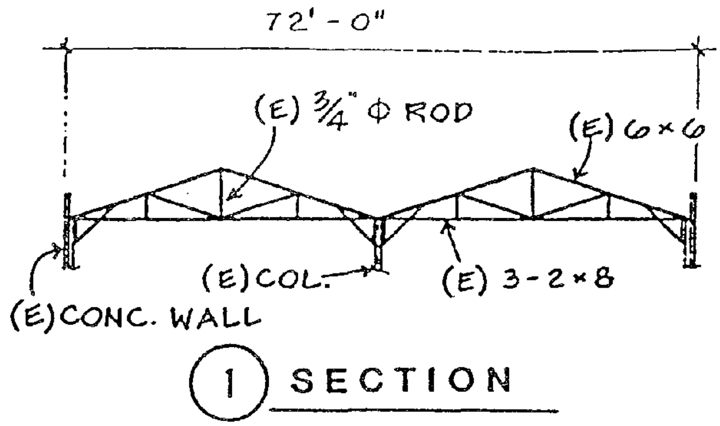
Roof Plan, Example 4 Building.

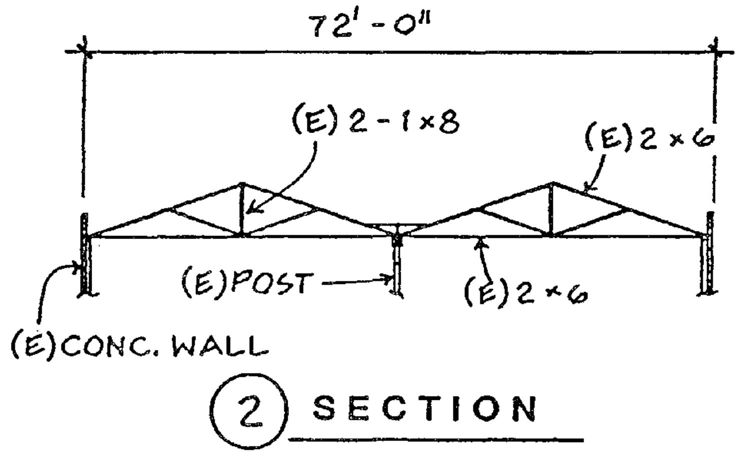
FIGURE 12.29

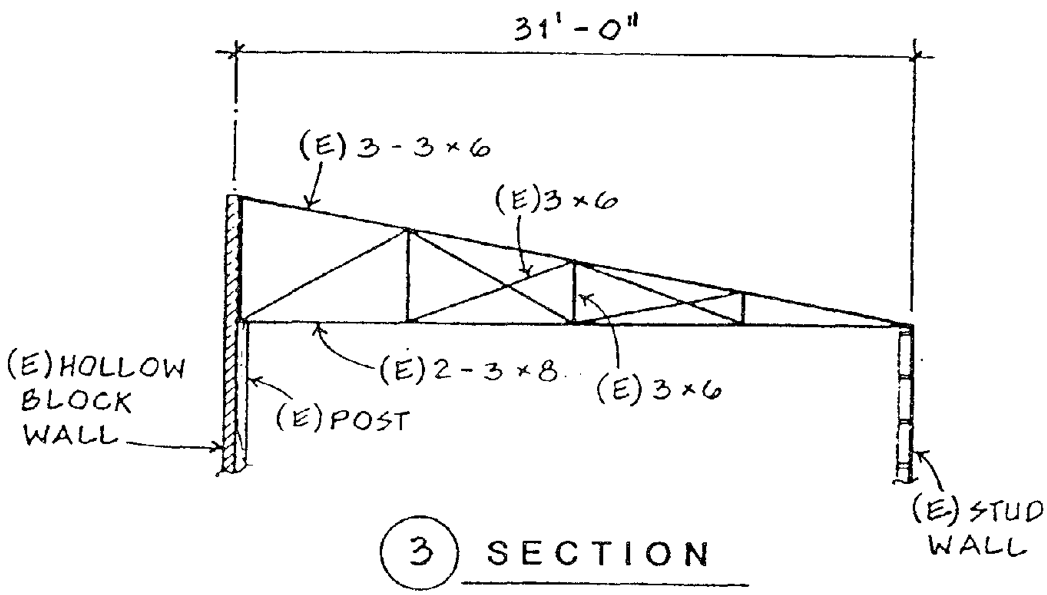


Typical Roof Framing in Building "A", Example 4 Building.

FIGURE 12.30







Sections Showing Various Roof Truss Framing Schemes, Example 4 Building.

FIGURE 12.31

ATC-14 Field Data Sheet

Building Data

Year built: ? Year(s) remodelled? ? Date: 10/12/86
Area,sf: 19000 Length 263 Width 72 Photo Roll #: 1
No. stories 1 Story height ______ Total ht. VARIES

Construction Data

Gravity load structural system : WOOD BEAMS OR TRUSSES ON POSTS OR CONC. WALL
Exterior transverse walls: CONC OR METAL SIDING Openings? —
Exterior longitudinal walls: CONC OR METAL SIDING Openings? ______
Roof materials/framing : STRAIGHT OR DIAGONAL SHEATHING - TRUSSES OR BEAMS
Intermediate floors/framing : —
Ground floor : CONC SLAB ON GRADE
Columns : WOOD POSTS Foundation : CONC. SLAB ON GRADE

Lateral Force Resisting System

	Longitudinal	Transverse
Diaphragms:	STRAIGHT OR DIAG. SHEATHING	SAME
Vertical Elements :	CONC. WALL - BLDG. "A" ONLY.	BLDG. "A" HAS CONC. WALLS OTHERS HAVE NO REAL LATERAL BRACING.
Connections :	SOME DIAGONAL BRACING IN "C"	
Details :		TRUSS DETAILS ARE POOR

Building Classification: W-2 - WOOD - COMMERCIAL

ATC-14 Checklist

ATC structural checklist completed and attached? YES
ATC non-struc " " " " ? NO. NO NON-STRUCTURAL

General condition of structural? FAIR Evidence of settling? NO
Special features/comments : 5 SEPARATE STRUCTURES WITH ROOF ELEVATIONS THAT VARY. WILL REQUIRE SEPARATE LATERAL SYSTEMS FOR EACH BUILDING. PRESENTLY ONLY BLDG "A" HAS A LATERAL WALL SYSTEM, AND IT IS C-SHAPED WITH A WOOD DIAPHRAGM.

Earthquake Damage Potential:

Limitted damage/ loss of function ( ) Minor damage/ loss of function ( )
• no repairs required • repairs while occupied
Major Damage/ loss of function (✓) Total Damage/ loss of function ( )
• repairs required prior to occupation • demolition

ATC-14 Field Data Sheet, Example 4 Building.

FIGURE 12.32

Checklist 1. Wood Buildings*
Type 1—Dwellings
Type 2—Commercial or Industrial

True/False			Comments
True	RAPID EVALUATION OF SHEAR STRESS IN WOOD OR GYPSUM WALLS REQUIRED		
	MATERIALS		
False	5.6.1	No signs of decay, sagging, splitting of wood, or deterioration of metal accessories.	Truss in Building "D", and some members in Building "B" trusses.
NA	5.6.2	No overdriven nails. No plywood walls	
	STRUCTURAL ELEMENTS		
False	5.6.3	Type 2—Diaphragms. No straight or diagonal sheathing, or span/depth ratios greater than 2.	Buildings "B" and "E" in transverse direction
True	5.6.4	No split level floors or expansion joints (Type 2).	
NA	5.6.5	Large openings (e.g., garage doors) are braced or tied-in.	
NA	5.6.6	Walls with tributary areas less than 100 ft^2 per foot of wall are plywood or rod-braced. If H/D > 1, have hold-downs.	
NA	5.6.7	Cripple walls are braced.	
NA	5.6.8	Walls are bolted to sill at 6 feet or less spacing.	
NA	5.6.9	Diaphragm openings greater than 50 percent of width have reinforcing.	
False	5.6.10	Diaphragms with greater than 24 foot span are plywood or rod braced (Type 2).	All buildings
	FOUNDATIONS		
True	5.6.11	Posts are positively connected to foundation.	
	NONSTRUCTURAL ELEMENTS		
True	5.6.12	No unreinforced masonry chimneys.	
NA	5.6.13	Reinforced masonry chimneys are tied into all diaphragms.	
NA	5.6.14	Masonry veneer above first floor positively attached at less than 2 feet on center.	

Note: Hollow block wall and parapet between Buildings "D" and "E" should be evaluated!

*See Chapter 5, Section 5.6, for detailed discussions on each of these checklist issues.

Structural Evaluation Checklist for Wood Buildings, Example 4 Building.

FIGURE 12.33

SECTION 13

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13.2 References Reviewed During Development of Methodology

This section lists all of the material that was encountered during the course of the literature survey and other aspects of the project. The references have been categorized and are listed under the following major topics: (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, and (12) historical references. Included at the beginning of each section is a description of the type of information collected and presented in that section. The references that were used in the development of the methodology and those that were read carefully, but not used directly, are listed at the start of each section (references receiving in-depth review), whereas those that received only a cursory review are included at the end (references receiving cursory review).

Earthquake Damage Reports

This section lists reports on the observations of damage caused by major earthquakes of this century. These reports provide insight into the causes of the performance characteristics that different types of building construction exhibit during earthquakes.

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Evaluation Methodologies and Examples

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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, 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 the NCEER Project Review Team. These references, including a brief description and the Library of Congress classification number, are listed below.

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

Brush, William D., Compiler, 1922, Building Code of New York City, 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.*

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

Earthquake and Building Construction, 1929, Clay Products Institute of California. *This book describes the response of masonry and masonry infill buildings in various earthquakes, and describes the use of masonry in building design to resist seismic forces.*

Ferris, Herbert W., 1983, 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, American Institute of Steel Construction, Chicago, Illinois. *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.*

Fowler, Charles Evans, 1897, General Specifications for Steel Roofs and Buildings, 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.*

Grinter, Linton E., 1941, Design of Modern Steel Structures, The MacMillan Company, New York. *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.*

Hatt, William K., and Voss, Walter C., 1921, Concrete Work: A Book to Aid the Self-Development of Workers in Concrete and for Students in Engineering, Volume 1, 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)

Hool, George A., and Kinne, W.S., 1923, Structural Members and Connections , 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.*

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

Hool, George A., and Johnson, Nathan C., Editors, 1929, Handbook of Building Construction, Volume I, 2nd Edition, 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.*

Jacoby, Henry S. and Davis, Roland P., Timber Design and Construction, 2nd Edition, 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.*

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

Kidder, F.E., Architect, 1900-1920, Building Construction and Superintendence, Part I - Mason's Work, 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.*

Kidder, F.E., Architect, 1900-1920, Building Construction and Superintendence, Part II - Carpenter's Work, 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.*

Kidder, F.E., Architect, 1906, Building Construction and Superintendence, Part III - Trussed Roofs and Roof Trusses. *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.*

Lord, Arthur R., 1929, Handbook of Reinforced Concrete Building Design, 1st Edition, 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".*

Modern Connectors for Timber Construction, 1933, National Committee on Wood Utilization, U.S. Department of Commerce, Government Printing Office, Washington, D.C. *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.*

Mulligan, John A., 1942, Handbook of Brick Masonry Construction, 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.).*

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.*

Scott, W.F., 1904, Structural Designers Handbook. *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.*

"Sweet's" Indexed Catalogue of Building Construction, 1906, Beginning in 1906, Architectural Record Company, New York. *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.*

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

APPENDIX A

ATC-14 PROJECT PARTICIPANTS

AND

NCEER REVIEW PROJECT PARTICIPANTS

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APPENDIX B

STATE-OF-PRACTICE REVIEW

QUESTIONNAIRE FOR DESIGN PROFESSIONALS

STATE OF PRACTICE REVIEW
QUESTIONNAIRE FOR DESIGN PROFESSIONALS

Design professionals were interviewed as part of the information search in the development of the ATC-14 methodology for evaluating the seismic strengths of existing buildings. These interviews were intended to determine the basic approach followed and any special techniques or procedures that have been used by the different structural offices.

The participants in the interview process were asked to complete a questionnaire concerning the methods that each firm uses to evaluate the seismic strength of existing buildings. While many of the questions could have been answered with either a simple "yes" or "no," each participant was encouraged to provide a short explanation to each response, when appropriate.

Following are the questions asked of each participants and a summary of the responses:

I. General Information

1. Approximately what proportion of your business deals with the evaluation and/or retrofit of existing buildings for seismic hazards?

   The range of responses varied from 1 percent to 75 percent. The most typical was 10 percent to 15 percent. As expected, the smallest proportions were from consultants in regions of lower seismicity.

   la. What type of clients want this information, and why?

   The responses included the following major groups:

   (1) Government or Public Agencies-Navy, Corps of Engineers, Defense Department, school systems, etc.;

   (2) Developers or private clients, such as hospitals or large corporations;

   (3) Architects.

2. Does your office have a written procedure for the evaluation of the seismic strength of existing buildings?

   Only one consultant had a written procedure for seismic evaluation. One other firm had a set of basic guidelines to follow, and another had used an evaluation form developed by an insurance company. Many of the consultants stated that because each of the buildings is unique, the evaluations should be tailored to each individual project. One reply stated that they could not justify the cost of developing such a procedure because the evaluations were such a small portion of their work.

2a. If so, are different types of constructions and/or materials treated separately?

Each consultant treats different buildings separately based on the type of construction or materials. The basic categories given were: steel frames, concrete frames, unreinforced masonry, wood, post-tensioned garages, precast frames, and reinforced masonry. The most common building type evaluated was unreinforced masonry.

2b. Is the office design procedure based on a previous methodology (e.g., Navy Rapid Seismic Analysis, General Service Administration Provisions, etc.)?

None of the firms used any of the previous methodologies unless required to do so by the client. Most were aware of the different methodologies and many had used the Navy Rapid Seismic Analysis Procedure. Some stated that they referred to the ABK Research (ABK, 1984) for their work with unreinforced masonry.

3. Have any of the techniques or research results of foreign work, such as in Japan or New Zealand, been incorporated into any of the evaluations?

Most of the consultants do not use any foreign research results in their evaluations. Many stated that they were aware of work done in Japan, New Zealand, Yugoslavia, and Italy. Code work done in Japan, New Zealand, and Europe was also mentioned as being useful. It was mentioned that it is difficult to use much of this material because of the differences in construction techniques.

4. Have any of the research results of United States investigations been used in any of the evaluations?

Many of the consultants mentioned that they use the results of the ABK work on unreinforced masonry buildings. Some try to use information on member force and deformation capacity from tests. Some work on nonlinear analysis has been reviewed.

5. What are the sources of this information? Do you feel there is a need to receive more information?

The sources of information given are:

1. Journals-American Society of Civil Engineers (ASCE), Earthquake Engineering Research Institute (EERI), and Building Science Researchers.

2. Reports-National Science Foundation (NSF), Applied Technology Council (ATC), etc. Only a few mentioned using direct research in their work. They would rather see the research distilled into more useful forms. Most felt that the information was not readily available. Some mentioned that seminars and workshops are good sources of information.

II. Information Gathering

1. What do field investigations to buildings under evaluation entail?

   The first step in field investigation work is usually to do a walk-through tour of the structure, looking for signs of distress. A large number of pictures are taken at this time. This tour is also used to determine which details, if any, need to be exposed. Some consultants stated that they tailor their field investigations to the type of building. Foundation and soil conditions are also checked. They try to determine typical connection details, nonstructural elements, and the basic structural system.

2. Are field tests done to measure material strengths?

   All the consultants typically do material tests of concrete and masonry to determine compressive and/or shear strengths. In-place shear tests of masonry are widely used, although cores are also used to determine the adequacy of inner wythes. Pile cores and soil tests were also mentioned as were tests of wall anchors. Steel coupons or rebar tests are done only in special cases. The strength of steel can be estimated fairly well from its age. Pacometer, ultrasonic, borescope, and petrographic tests have also been used by some of the consultants.

3. If structural drawings are available, are they verified by site investigations?

   All of the consultants do some verification of existing drawings. For newer buildings, verifying the structural drawings may not be a complicated process. For older buildings, the drawings that exist may not be complete. Most consultants try not to do any destructive exploration unless it is absolutely necessary for a critical element.

4. If no structural drawings are available, are drawings prepared from field investigations?

   This case is felt to be typical of older buildings, especially of unreinforced masonry. Most of the consultants do prepare some sketches or drawings. Field measurements are often done to lay out the basic structural elements. The critical elements and connections will be examined more closely. The structure is exposed more in this case so that the "as-built" drawings can be produced.

5. If calculations are available, are they used in the evaluation procedure?

   Most, but not all, of the consultants, will use the original calculations if they are available. The basic information to be drawn from the calculations are the base shear level of the original design and the basic assumptions of the design. After performing their own analysis, many of the consultants said they check the original calculations to determine how the engineer dealt with trouble spots. It was mentioned that the calculations should be checked with both the drawings and the construction. Some of the consultants said they will not look at the original calculations until after they have done their own analysis, so that the results will not be biased by the original design.

III. Preliminary Analysis Techniques

1. Is the first phase of the evaluation procedure basically a qualitative review of typical building characteristics?

   Most of the consultants use a primarily qualitative analysis for the preliminary evaluation stage, although some rough calculations are usually done. The details are evaluated for their ability to tie the structure together. The basic structural system and possible life-safety hazards are identified.

2. If calculations are performed in the preliminary evaluation phase, what are they?

   Rough estimates of building weights and distribution of loads to lateral force resisting elements are preliminary calculations. Basic shear stress values are determined, typically at code force levels. They use these values to calibrate their judgment. At this level, they are concerned with order of magnitude values of stresses. Generally, only hand calculations are done at this phase.

3. Is the building given a numerical index to rate the expected seismic performance?

   None of the consultants give the building a numerical index unless they are required to do so by the client. Many of the responses included a statement that they thought such indexes were a "numbers game." One consultant stated that such indexes miss the problems caused by the "fatal flaw."

4. How do you decide if the building is adequate or if it needs further study?

   Most of the consultants base their recommendations on their experience from the performance of different building types in past earthquakes. They temper their judgment with the preliminary calculations. Many of the consultants are reluctant to say certain building types are adequate without more detailed evaluations. Buildings with poor configurations and/or improper proportions will always trigger more detailed evaluation. One consultant will pass on a building if the expected damage level is the same as a 1955-1976 vintage structure with no life-safety hazards. Basically, this decision is based on judgment based on the consultants' experience.

IV. Detailed Analytical Techniques

1. Are the lateral loads used for the evaluation based on code level forces (Uniform Building Code (UBC), ATC, etc.)?

   Most of the consultants use code level forces for their evaluations. Some said that this was a requirement of the local jurisdiction. Others will sometimes use a fraction of the code forces, possibly from 50 to 70 percent of the code level. For unreinforced masonry, the basic reference for many of the consultants is Division 88 of the Los Angeles Building Code. Some of the consultants will factor the code level forces up to determine the "actual" exposure of the structure to the ultimate capacities. Factors ranging from 2 to 8 were mentioned, with the most typical value being about 3.

2. If the lateral loads used for the evaluation are not based on code level forces, are site specific response spectra used? What level of overall building ductility, if any, is used?

   Site specific response spectra are not typically used for existing buildings, except in some cases for large or important structures. In lieu of a site specific spectrum, ATC-3-06 spectra (ATC, 1978) are often employed. The ductility factors are dependent on the building type. Some consultants calculate the overall building ductility demand to see if it is reasonable. Others choose a reduction factor that will result in loads near code level forces. For some building types, such as R/C frames, some consultants do not reduce the response spectrum, but instead perform a nonlinear analysis that follows the change in period and damping as hinge formation occurs.

3. Have time-history analyses ever been employed in any of your evaluations?

   None of the consultants have used time-history analyses for the evaluation of existing buildings. It was felt that little information would be gained from such analyses.

4. Are code provisions used as the basis for evaluating member capacity? If so, are they based on the UBC, or on some other code?

   Code provisions are used to evaluate member capacities by most of the consultants. Research results are used by some of the engineers for items where the code equations are questionable, such as shear stress in concrete. Some consultants use actual, rather than nominal, values for yield stress. It was stated that present code equations may not be directly applicable to older types of construction. The different properties of older materials must be considered.

5. Are the capacities of secondary lateral force resisting systems included in the determination of the building strength?

   Most of the consultants replied that they will use secondary elements for certain types of building systems. For instance, in wood houses, including the capacities of plaster or gypboard walls is common. Some consultants stated that they will only use secondary elements that are permanent. The interaction of the secondary system with the primary elements is the important consideration. Some of the consultants stated that they will only use these elements if they can contribute significantly to the strength of the structure.

6. Are the connection capacities checked, including the estimated ductility?

   Approximately half of the consultants calculate the capacity of the connections. The other half typically checks the connections for the code level forces, possibly with a factor of 1.25. None of the consultants estimate the ductility of connections, although some stated that they use their judgment on the connection ductility to estimate a reduction factor for the structure. The details are checked to be sure that redistribution can occur.

7. Are non-linear modeling techniques ever incorporated in your analyses?

Most of the consultants do not use any non-linear analysis techniques in their evaluations. Those that do perform a step-wise linear analysis that follows the change in period and load level as the hinges form. The amount of rotation required at the hinges is checked. This analysis procedure is used for concrete frame buildings with response spectrum loadings.

8. What drift limitations do you apply? Are different limits used for different building types?

The usual draft limit applied is the code value of .005. Most consultants consider drift only for frame type buildings. Some do not consider drift at all for existing buildings because they are generally low rise. It was felt that this concern is more for nonstructural elements. Concentrations of drift in a single story should be examined more closely. The displacement values given by modal analysis may be too high. Stability Is the concern for structural elements.

9. How do you analyze vertical and/or plan irregularities?

Chord stresses are analyzed and the continuity of the load path is checked. Most feel that consideration of irregularities is important because they can be a cause of potential failure. Vertical strength irregularities are more important than stiffness irregularities, as changes in stiffness do not change the distribution significantly. One consultant felt that plan irregularities can reduce the base shear. Torsional displacements caused by open fronts must be considered in the stability of the gravity load system.

10. How do you decide if the building is adequate?

Many of the consultants stated that their decision on the adequacy of the building was still based on their judgment. Others stated that if there was sufficient base shear capacity, good connections, and a balanced system, they could say a building was adequate. Experience gained from past performance seems to be the basis of the judgments.

V. Other Items

1. Have you ever performed damage estimates as a result of the building evaluations?

Almost all of the consultants have done some sort of damage evaluation in the past. Most are general discussions of the type of damage that can occur for the building type being reviewed. Most of the engineers said they try to avoid making actual cost estimates. If pressed to give a dollar estimate, many said they give a range of values, or a percentage of construction cost.

2. Have you directly employed probabilistic methods in your building evaluations?

   Most of the consultants do not directly apply probabilistic methods in their evaluations. One consultant felt that these methods may be more applicable for large inventory studies than single building evaluations. This consultant also felt that probabilistic methods are better suited to estimating damage than to predicting life-safety performance.

3. Have you included nonstructural items in your evaluations?

   All of the consultants have included nonstructural items in their evaluations. Evaluation of nonstructural items is generally performed during walk-through tours of the building. Most of the recommendations concern life-safety hazards that can be caused by the following items: ceilings, lights, mechanical equipment, heavy cabinets and furniture, pipes, and windows.

4. Does the evaluation procedure conclude with a detailed written report of your findings?

   All of the consultants said that their evaluations typically conclude with a written report. Sometimes forms are filled out for insurance clients.

5. If strengthening is required, to what extent, and at what level do you recommend?

   The minimum level for strengthening was agreed to be the elimination of life-safety hazards. Some of the consultants stated that the local building official will often require a level of strengthening to the present or some past code value. The consideration of the occupancy and expected life span of the building were also mentioned as factors that influence the recommended level of strengthening. Some consultants tell the owner both the minimum level that is required for life safety, and a higher level that is necessary for damage control.

APPENDIX C

FIELD DATA SHEET AND ABBREVIATED EVALUATION CHECKLISTS FOR MODEL BUILDING TYPES IN AREAS OF LOW SEISMICITY

ATC-14 FIELD DATA SHEET

The ATC-14 Field Data Sheet (following page) was developed for use during the initial site visit and data collection portions of the building evaluation. This form is intended to identify all of the basic information about the building that is needed to begin the evaluation. The form is subdivided into six major categories:

1. Building Data - provides the general information about the age and size of the building.

2. Construction Data - identifies the gravity load resisting system, exterior walls, and construction materials used in the building.

3. Lateral Force Resisting System - identifies the diaphragm and vertical elements of the lateral force resisting system.

4. Building Classification - identifies the building type in terms of the model building types developed for this methodology (this classification is based on the above information).

5. ATC-14 Checklist - checks that the appropriate structural and non-structural checklist forms have been completed and provides an opportunity to comment on the general condition of the building, evidence of settling, and any other issues deemed significant but not addressed in the checklists.

6. Earthquake Damage Potential - is intended to allow the evaluating engineer to categorize his/her first impression of the seismic resistance of the building. This section can also be useful for prioritization when a large number of buildings are being evaluated.

This data sheet should be completed along with the appropriate ATC-14 structural and nonstructural evaluation checklists during the earliest stages of the seismic evaluation. See Chapter 12 for use of this form in the four example building evaluations.

STRUCTURAL AND NONSTRUCTURAL EVALUATION CHECKLIST

The structural evaluation checklists for each model building type and the nonstructural evaluation checklist are provided in this appendix on the pages following the ATC-14 Field Data Sheet.

ATC-14 Field Data Sheet

Building Data

Year built:_____ Year(s) remodelled?________ Date:______________
Area,sf: _______ Length ______ Width ______ Photo Roll #:________
No. stories ______ Story height _______ Total ht. _______

Construction Data

Gravity load structural system : ________________________________
Exterior transverse walls: ____________________________ Openings? _________
Exterior longitudinal walls: ___________________________ Openings? _________
Roof materials/framing : ______________________________________
Intermediate floors/framing : ___________________________________
Ground floor : ____
Columns : _____________________________ Foundation : ____________________

Lateral Force Resisting System

	Longitudinal	Transverse
Diaphragms: Vertical Elements : Connections : Details :		

Building Classification ________________________

ATC-14 Checklist

ATC structural checklist completed and attached? ______________________
ATC non-struc " " " " ? ______________________

General condition of structural? __________________ Evidence of settling? _____
Special features/comments :

Earthquake Damage Potential

Limitted damage/ loss of function ( )
• no repairs required

Minor damage/ loss of function ( )
• repairs while occupied

Major Damage/ loss of function ( )
• repairs required prior to occupation

Total Damage/ loss of function ( )
• demolition

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 (i.e. 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/
False | | Comments

NON-STRUCTURAL ELEMENTS

___ 5.5.11 Exterior cladding and veneer are well anchored.

___ 5.5.12 Reinforced masonry chimneys are tied into all diaphragms.

*See 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 Chapter 6, Section 6.1.5 for detailed discussion on each of these checklist issues.

Checklist 3. Braced Steel Frame Buildings*
(Low Seismicity Regions)

True/ False			Comments
	MATERIALS		
___	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.	
	STRUCTURAL ELEMENTS		
___	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
	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 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 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 A_g 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 Chapter 6, Section 6.4.5 for detailed discussion on each of these checklist issues.

Checklist 6. Steel Frame Buildings With Infilled Walls of Unreinforced Masonry*
(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

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 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

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 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.	

True/ False			Comments
___	7.2.5.13	Reinforcing in each diaphragm to transfer loads to walls.	
___	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 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.	

True/
False

Comments

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 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 A_g each way with a maximum spacing of 18 inches.	
___	8.1.5.12	No torsion.	

True/ False			Comments
___	8.1.5.13	Metal deck has topping slab with a minimum thickness of 3 inches.	
___	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 Chapter 8, Section 8.1.5 for detailed discussion on each of these checklist issues.

Checklist 11. <u>Precast Concrete Frame and Concrete Shear Wall Buildings</u>*
(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 A_g 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.	

True/ False			Comments
___	8.2.5.13	Metal deck has topping slab with a minimum thickness of 3 inches.	
___	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.	

True/ False			Comments
	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 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 Mortar Quality - Cannot scrape with metal tool and no large areas of eroded mortar.

___ 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 0.002 A_g with 0.0007 A_g minimum in either direction. Maximum spacing 48 inches. All vertical bars extend to the top of the wall.

True/
False | | | Comments

___ 9.1.5.12 No torsion.

___ 9.1.5.13 No vertical mass or geometric irregularities.

___ 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 Chapter 9, Section 9.1.5 for detailed discussion on each of these checklist issues.

Checklist 13. Reinforced Masonry Bearing Wall Precast Concrete Diaphragm Buildings*
(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	Mortar Quality - Cannot scrape with metal tool, and no large areas of eroded mortar.	
___	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 0.002 A_g with 0.0007 A_g 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
___	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		
___	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		
___	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 Chapter 9, Section 9.2.5 for detailed discussion on each of these checklist issues.

Checklist 14. Unreinforced Masonry Bearing Wall Buildings*
(Low Seismicity Regions)

True/ False			Comments
	RAPID EVALUATION OF SHEAR STRESS IN MASONRY WALLS REQUIRED		
	MATERIALS		
___	10.5.1	No signs of significant deterioration in vertical or lateral force resisting system.	
___	10.5.2	Mortar Quality - Cannot scrape with metal tool and no large areas of eroded mortar.	
___	10.5.3	No substantial leakage damage to roof deck.	
___	10.5.4	No damage to masonry and/or concrete elements due to freeze/thaw action.	
___	10.5.5	No damage to concrete surfaces due to chloride-laden concrete.	
	STRUCTURAL ELEMENTS		
___	10.5.6	Complete lateral force resisting system forming a continuous load path and tieing all portions of the building together.	
___	10.5.7	Building has a redundant lateral system insuring structural stability in the event of a single component failure.	
___	10.5.8	No vertical strength discontinuities.	
___	10.5.9	No torsion.	
___	10.5.10	No vertical mass or geometric irregularities.	
___	10.5.11	All walls are continuous to the foundation.	
___	10.5.12	No pounding of adjacent structures.	

True/
False

Comments

___ 10.5.13 Masonry walls are attached to wood diaphragms with steel anchors or straps that are connected to a diaphragm cross tie.

___ 10.5.14 Wall anchors spaced at 4 feet or less.

___ 10.5.15 Gable ends are anchored at all diaphragm levels.

___ 10.5.16 No openings adjacent to masonry walls larger than 8 feet.

___ 10.5.17 Diaphragm openings at walls are less than 25 percent of the length.

___ 10.5.18 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.

___ 10.5.19 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.

___ 10.5.20 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.

FOUNDATIONS

___ 10.5.21 In a pile foundation, the lateral stiffness and strength below grade is at least that of above grade.

___ 10.5.22 The foundation is not composed of unreinforced masonry or stone rubble.

___ 10.5.23 There is no foundation or superstructure damage due to heaving soil.

___ 10.5.24 Buildings taller than six stories are not founded on a soil which is subject to liquefaction.

True/ False			Comments
	NON-STRUCTURAL ELEMENTS		
___	10.5.25	Cornices, parapets, and other appendages are reinforced and anchored to the structure.	
___	10.5.26	Exterior cladding and veneer are well anchored.	
___	10.5.27	No unreinforced masonry chimneys.	
___	10.5.28	Reinforced masonry chimneys are tied into all diaphragms.	

*See Chapter 10, Section 10.5 for detailed discussion on each of these checklist issues.

Checklist 15. Nonstructural Elements*

True/ False			Comments
	PARTITIONS		
___	11.2.1.1	All unreinforced masonry partitions are less than 8 feet tall. (LS)	
___	11.2.1.2	Partitions are detailed to accommodate interstory drift.	
___	11.2.1.3	No partitions cross seismic joints.	
___	11.2.1.4	Partitions that only extend to ceiling are laterally braced at the top.	
	FURRING		
___	11.2.2	No structural elements are furred.	
	CEILINGS		
___	11.2.3.1	No suspended plaster or gyp board ceilings.	
___	11.2.3.2	No clips used to attach ceiling panels or tiles.	
___	11.2.3.3	No lay-in tiles are used.	
___	11.2.3.4	No ceilings cross seismic joints.	
___	11.2.3.5	Ceilings are not required to provide lateral bracing of partitions.	
___	11.2.3.6	Ceiling edges are separated from structural walls.	
	LIGHT FIXTURES		
___	11.2.4.1	Multiple length fluorescent fixtures are laterally braced or have secondary support.	
___	11.2.4.2	Fluorescent light fixture lenses have safety chains or a positive attachment.	

True/ False			Comments
___	11.2.4.3	Pendant fixtures can swing without contacting other elements.	
___	11.2.4.4	No double stem fluorescent fixtures are used.	
	MECHANICAL EQUIPMENT		
___	11.2.5.1	Large equipment is positively attached to the structural system.	
___	11.2.5.2	Vibration isolated equipment has restraints to limit motion.	
___	11.2.5.3	No major equipment is suspended from the ceiling without seismic bracing. (LS)	
	PIPING		
___	11.2.6.1	All pipes crossing seismic joints have flexible connectors.	
___	11.2.6.2	No pipes support other pipes.	
___	11.2.6.3	All pipe sleeve wall openings are at least 2 in. larger than the pipe.	
	DUCTS		
___	11.2.7.1	Long duct lines are laterally braced.	
___	11.2.7.2	No ducts are supported by pipes or other non-structural elements.	
___	11.2.7.3	All ducts crossing seismic joints have flexible connections.	

True/
False

Comments

ELECTRICAL EQUIPMENT

___ 11.2.8.1 All electrical equipment is positively attached to the structure.

___ 11.2.8.2 Rigid conduits or bus ducts have flexible sections at seismic joints.

ELEVATORS

___ 11.2.9.1 Elevator support system is adequately anchored to resist lateral forces. (LS)

___ 11.2.9.2 Guide rails can support horizontal forces between support brackets for C_p = .30. (LS)

___ 11.2.9.3 Cable retainer guards on sheaves and drums properly protect cable displacement.

___ 11.2.9.4 Snag points have guards to prevent snagging of relevant moving elements. (LS)

___ 11.2.9.5 Clearance between the car and counterweight assembly and between the counterweight assembly and the hoist way enclosure is greater than 2 inches. (LS)

___ 11.2.9.6 Maximum spacing of counterweight rail tie brackets is 16 feet. Intermediate spreader brackets are provided for spans greater than 3/16 inches.

___ 11.2.9.7 Retainer plates are provided at the top and bottom of both the car and the counterweight, with clearances greater than 3/16 inches.

___ 11.2.9.8 Control panels are bolted to the floor slabs.

CLADDING, GLAZING AND VENEER

Materials

___ 11.2.10.1a No substantial damage to cladding due to water leakage.

___ 11.2.10.1b No damage to cladding due to temperature movements.

True/ False			Comments
	Brick Veneer With Concrete Block Backup		
___	11.2.10.2a	Brick veneer is supported by shelf angles or other elements at each floor level. (LS)	
___	11.2.10.2b	Brick veneer is adequately anchored to the backup at locations of through-wall flashing. (LS)	
___	11.2.10.2c	Brick veneer is connected to the backup with ties at 24 inches o.c. maximum, with one tie every 2-2/3 foot square maximum. (LS)	
___	11.2.10.2d	Concrete block backup qualifies as reinforced masonry (high seismicity only). (LS)	
___	11.2.10.2e	Concrete block backup is positively anchored to the structural frame at 4'-0" maximum. (LS)	
___	11.2.10.2f	For moment frame buildings of steel or concrete (Sections 6.1 or 7.1), panels can accommodate an interstory drift of three inches. (LS)	
___	11.2.10.2g	Eccentricities in connections are accounted for. (LS)	
___	11.2.10.2h	Connections were installed generally in accordance with the drawings. (LS)	
___	11.2.10.2i	Connections are not deteriorated or corroded. (LS)	
___	11.2.10.2j	No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)	
___	11.2.10.2k	No cracking in the brick veneer indicative of structural distress. (LS)	
___	11.2.10.2l	Mortar joints in brick and block wythes are well-filled, and material cannot be easily scraped from the joints. (LS)	
	Brick Veneer with Steel Stud Backup		
___	11.2.10.3a	Brick veneer is supported by shelf angles or other elements at each floor level. (LS)	

True/ False			Comments
___	11.2.10.3b	Brick veneer is adequately anchored to the backup in the vicinity of locations of through-wall flashing. (LS)	
___	11.2.10.3c	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)	
___	11.2.10.3d	For moment frame buildings of steel or concrete (Sections 6.1 or 7.1), panels can accommodate an interstory drift of three inches without collapse. (LS)	
___	11.2.10.3e	Corrugated brick ties are not used. (LS)	
___	11.2.10.3f	Eccentricities in connections are accounted for. (LS)	
___	11.2.10.3g	Connections were installed generally in accordance with the drawings.	
___	11.2.10.3h	Connections are not deteriorated or corroded. (LS)	
___	11.2.10.3i	No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)	
___	11.2.10.3j	No cracking in the brick veneer indicative of structural distress. (LS)	
___	11.2.10.3k	Computed tensile stresses in the veneer do not exceed the allowable. (LS)	
___	11.2.10.3l	Mortar joints in the brick veneer are well filled, and material cannot be easily scraped out from the joints. (LS)	
___	11.2.10.3m	Additional steel studs frame window and door openings. (LS)	
___	11.2.10.3n	No visible corrosion of brick ties, tie screws, studs, or stud tracks. (LS)	
___	11.2.10.3o	No visible deterioration of exterior sheathing. (LS)	
___	11.2.10.3p	Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)	

True/
False

Comments

Precast Concrete

___ 11.2.10.4a There are at least two bearing connections for each wall panel. (LS)

___ 11.2.10.4b There are at least four connections for out-of-plane forces. (LS)

___ 11.2.10.4c Multi-story panels are attached at each floor level. (LS)

___ 11.2.10.4d For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels can accommodate an interstory drift of three inches. (LS)

___ 11.2.10.4e Inserts used to attach wall elements are welded to or hooked around reinforcing steel. (LS)

___ 11.2.10.4f Eccentricities in connections are accounted for. (LS)

___ 11.2.10.4g Connections with welded inserts appear to be capable of yielding in the base metal before fracture. (LS)

___ 11.2.10.4h Connections were installed generally in accordance with the drawings. (LS)

___ 11.2.10.4i Connections are not deteriorated or corroded. (LS)

___ 11.2.10.4j No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)

Thin Stone Veneer Panels

___ 11.2.10.5a There are at least two bearing connections for each wall panel. (LS)

___ 11.2.10.5b There are at least four connections for out-of-plane forces. (LS)

___ 11.2.10.5c For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, panels and connections can accommodate an interstory drift of three inches. (LS)

True/ False			Comments
___	11.2.10.5d	For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels can accommodate an interstory drift of three inches. (LS)	
___	11.2.10.5e	Inserts used to attach wall elements are welded to or hooked around reinforcing steel. (LS)	
___	11.2.10.5f	Eccentricities in connections are accounted for. (LS)	
___	11.2.10.5g	Connections with welded inserts appear to be capable of yielding in the base metal before fracture. (LS)	
___	11.2.10.5h	Connections were installed generally in accordance with the drawings. (LS)	
___	11.2.10.5i	Connections are not deteriorated or corroded. (LS)	
___	11.2.10.5j	No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)	
___	11.2.10.5k	Stone anchorages are adequate for computed loads. (LS)	
___	11.2.10.5l	no visible cracks or weak veins in the stone. (LS)	
	Glass and Metal Curtainwall Panels		
___	11.2.10.6a	There are at least two bearing connections for each curtain wall panel. (LS)	
___	11.2.10.6b	There are at least four connections for out-of-plane forces. (LS)	
___	11.2.10.6c	For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, panels and connections can accommodate an interstory drift of three inches. (LS)	
___	11.2.10.6d	For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels can accommodate an interstory drift of three inches. (LS)	

True/
False

Comments

___ 11.2.10.6e Eccentricities in connections are accounted for. (LS)

___ 11.2.10.6f Connections with welded inserts appear to be capable of yielding in the base metal before fracture. (LS)

___ 11.2.10.6g Connections were installed generally in accordance with the drawings. (LS)

___ 11.2.10.6h Connections are not deteriorated or corroded. (LS)

___ 11.2.10.6i Inserts used to attach curtain wall elements are welded to or hooked around reinforcing steel. (LS)

Wood/Aggregate Panels

___ 11.2.10.7a There are at least two bearing connections for each wall panel. (LS)

___ 11.2.10.7b There are at least four connections for out-of-plane forces. (LS)

___ 11.2.10.7c For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, panels and connections can accommodate an interstory drift of three inches. (LS)

___ 11.2.10.7d For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels can accommodate an interstory drift of three inches. (LS)

___ 11.2.10.7e Eccentricities in connections are accounted for. (LS)

___ 11.2.10.7f Connections were installed generally in accordance with the drawings. (LS)

___ 11.2.10.7g Connections are not deteriorated or corroded. (LS)

___ 11.2.10.7h No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)

___ 11.2.10.7i Additional steel studs frame window and door openings. (LS)

True/ False			Comments
___	11.2.10.7j	No visible corrosion of tie screws, studs, or stud tracks. (LS)	
___	11.2.10.7k	No visible deterioration of exterior sheathing. (LS)	
___	11.2.10.7l	Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)	
___	11.2.10.7m	No visible deterioration of screws or wood at panel attachment points. (LS)	

Stucco Finish on Lath Panels

___	11.2.10.8a	For moment frame buildings of steel or concrete (Sections 6.1 and 7.1), where multi-story panels are attached at each floor level, panels and connections can accommodate an interstory drift of three inches. (LS)	
___	11.2.10.8b	For moment frame buildings of steel or concrete (Sections 6.1 and 7.1), panels can accommodate an interstory drift to three inches. (LS)	
___	11.2.10.8c	Eccentricities in connections are accounted for. (LS)	
___	11.2.10.8d	Connections were installed generally in accordance with the drawings. (LS)	
___	11.2.10.8e	Connections are not deteriorated or corroded. (LS)	
___	11.2.10.8f	No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)	
___	11.2.10.8g	Additional steel studs frame window and door openings. (LS)	
___	11.2.10.8h	No visible corrosion of tie screws, studs, or stud tracks. (LS)	
___	11.2.10.8i	No visible deterioration of exterior sheathing. (LS)	
___	11.2.10.8j	Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)	

True/
False

Comments

Composite Expanded Polystyrene and Stucco Panels

___ 11.2.10.9a There are at least two bearing connections for each wall panel. (LS)

___ 11.2.10.9b There are at least four connections for out-of-plane forces. (LS)

___ 11.2.10.9c For moment frame buildings of steel or concrete (Section 6.1 or 7.1) where multi-story panels are attached at each floor level, panels and connections can accommodate an interstory drift of three inches. (LS)

___ 11.2.10.9d For moment frame buildings of steel or concrete (Section 6.1 or 7.1), panels can accommodate an interstory drift of three inches. (LS)

___ 11.2.10.9e Connections were installed generally in accordance with the drawings. (LS)

___ 11.2.10.9f Connections are not deteriorated or corroded. (LS)

___ 11.2.10.9g No signs of leakage inside the building that indicates internal deterioration of the wall. (LS)

___ 11.2.10.9h Additional steel studs frame window and door openings. (LS)

___ 11.2.10.9i No visible corrosion of tie screws, studs, or stud tracks. (LS)

___ 11.2.10.9j No visible deterioration of exterior sheathing. (LS)

___ 11.2.10.9k Stud tracks are fastened to the structural frame at 24 inches o.c. maximum. (LS)

PARAPETS, CORNICES, ORNAMENTATION, AND APPENDAGES

___ 11.2.11.1 All unreinforced masonry parapets have h/t ratios less than 1.5. (LS)

___ 11.2.11.2 Al reinforced masonry parapets have h/t ratios less than 30. (LS)

True/ False			Comments
___	11.2.11.3	Concrete parapets with h/t ratios greater than 1.5 are reinforced. (LS)	
___	11.2.11.4	Appendages, cornices and other exterior wall ornamentations are anchored to the structure. (LS)	
	MEANS OF EGRESS		
___	11.2.12.1	Walls around stairs and corridors are not hollow tile or unreinforced masonry. (LS)	
___	11.2.12.2	All veneers, parapets, cornices, canopies, etc., above exits are well anchored. (LS)	
___	11.2.12.3	Lay-in ceiling tiles are not used in exits or corridors. (LS)	
	BUILDING CONTENTS AND FURNISHINGS		
___	11.2.13.1	Desk-top equipment is anchored to restrain it from sliding.	
___	11.2.13.2	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.	
___	11.2.13.3	Tall, narrow (H/D > 3) storage racks are anchored to the floor slab or adjacent walls. (LS)	
___	11.2.13.4	Plants, artwork and other objects are anchored to restrict their motion.	
___	11.2.13.5	Breakable items stored on shelves are restrained from falling by latched doors, shelf lips, wires, or other methods.	
___	11.2.13.6	Computers and Communications equipment are anchored to the floor slab and/or structural walls to resist overturning forces. (LS)	
___	11.2.13.7	Computer access floors are braced to resist lateral forces.	

True/ False			Comments
	HAZARDOUS MATERIALS		
___	11.2.14.1	Compressed gas cylinders are restrained against motion. (LS)	
___	11.2.14.2	Laboratory chemicals stored breakable containers are restrained from falling by latched doors, shelf lips, wires or other methods. (LS)	
___	11.2.14.3	Piping containing hazardous materials is provided with shut-off valves or other devices to prevent major spills or leaks. (LS)	

APPENDIX D

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 Appendix 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 3.6.2.6, 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 D.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 D.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:

$$\tau = 0.65\, a_{max}\, \sigma_o\, r_d \qquad \text{(D.1)}$$

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):

$$r_d = 1 - 0.015d \tag{D.2}$$

Equation D.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 D.3:

$$CSR = \frac{\tau}{\sigma_o} \tag{D.3}$$

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 D.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 D.1 to the cyclic stress ratio for the M_s = 7.5 (After Seed and De Alba, 1986).

TABLE D.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:

$$N_c = N \times ER \times C_n \qquad \text{(D.4)}$$

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 D.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 D.2.

TABLE D.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 D.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.

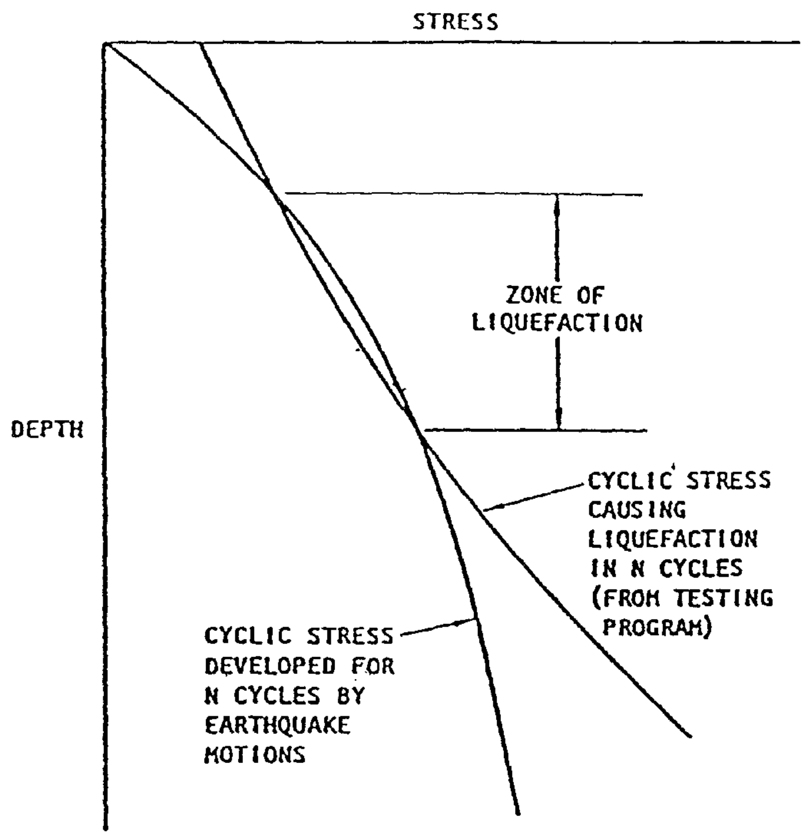

Figure D.1 - Method of Evaluating Liquefaction Potential
(Seed and Idriss, 1982)

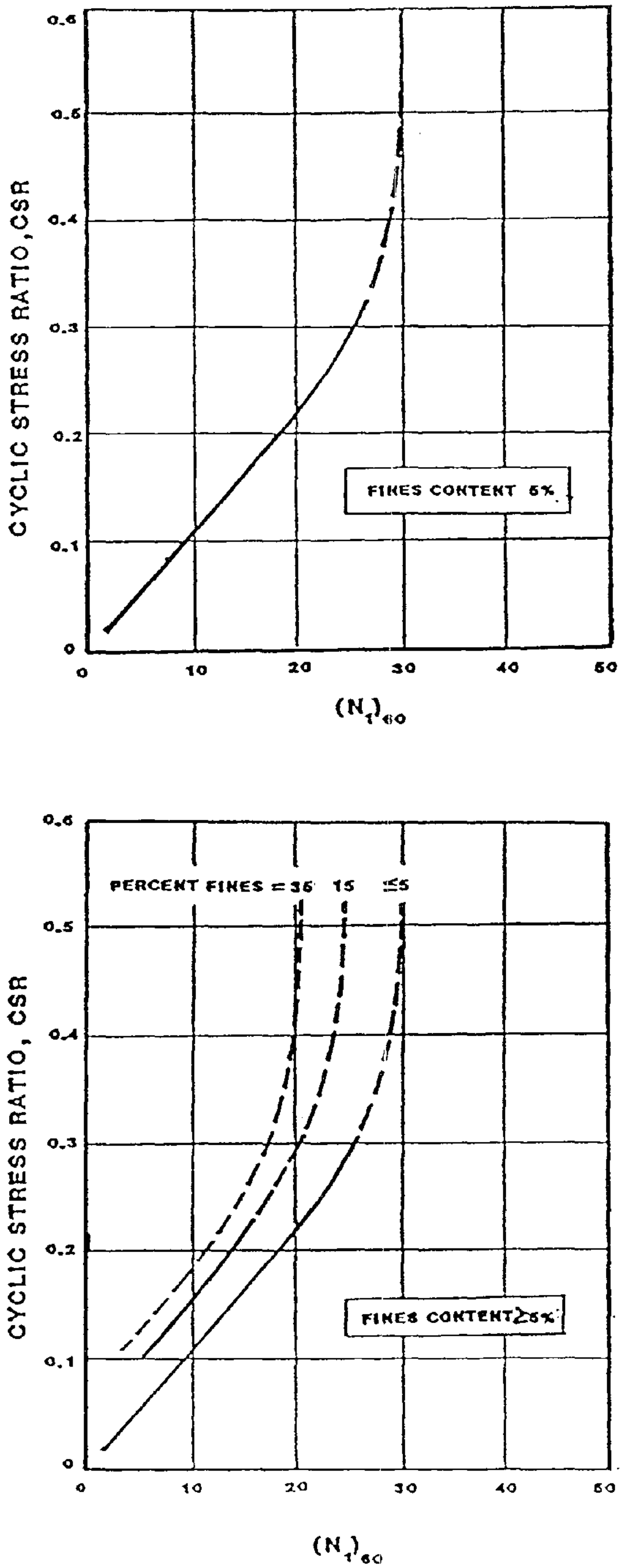

Figure D.2 - Empirical Relationship Between Shear Stress Ratio and SPT (Seed and De Alba, 1986)

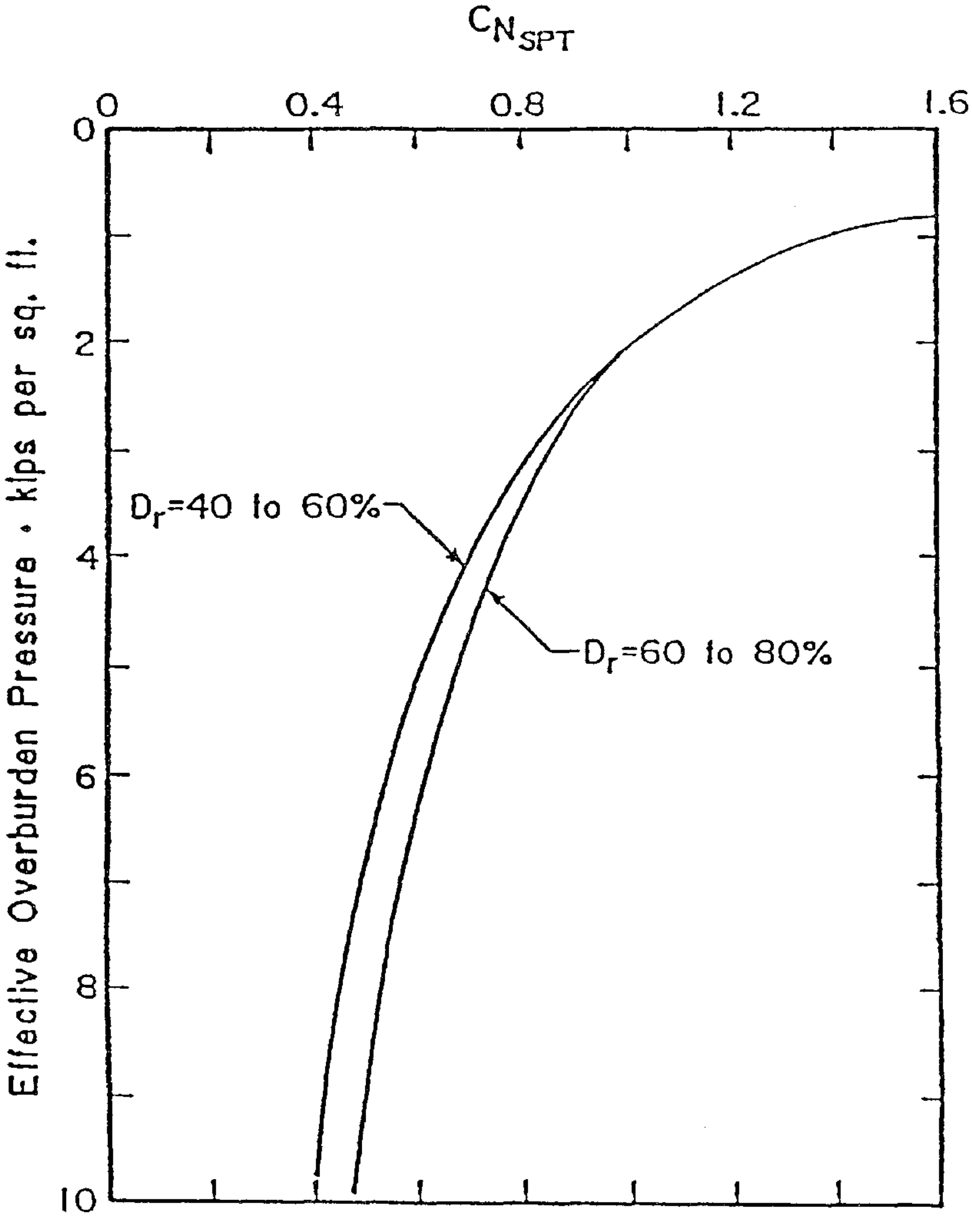

Figure D.3 - Overburden Correction Factor for N-Value (Marcuson and Bieganousky)

APPENDIX E

TESTING PROCEDURES

FOR THE SPECIAL EVALUATION

PROCEDURE OF SECTION 4.4.6

The following procedure was presented in the draft model ordinance of the California Seismic Safety Commission (SSC, 1985). See also UBC Standards 24-40 and 24-41.

E.1 Wall Tests

General Provisions. All unreinforced masonry wall utilized to carry vertical loads and seismic forces parallel and perpendicular to the wall plane shall be tested as specified in this subsection. All masonry quality shall equal or exceed the minimum standards established herein or shall be removed and replaced by new materials. Alternate methods of testing may be approved by the building official. The quality of mortar in all masonry walls shall be determined by performing in-place shear tests. Nothing shall prevent pointing with mortar of all the masonry wall joints before the tests are first made. Prior to any pointing, the mortar joints must be raked and cleaned to remove loose and deteriorated mortar. Mortar for pointing shall be Type S or N except masonry cements shall not be used. All preparation and mortar pointing shall be done under the continuous inspection of a registered deputy building inspector. At the conclusion of the inspection, the inspector shall submit a written report to the licensed engineer or architect responsible for the seismic analysis of the building, setting forth the result of the work inspected. Such report shall be submitted to the building official for approval as part of the structural analysis. All testing shall be performed in accordance with the requirements specified in this subsection by a testing agency approved by the building official. An accurate record of all such tests and their location in the building shall be recorded and these results shall be submitted to the building official for approval as part of the structural analysis.

Number and Location of Tests. The minimum number of tests shall be two per wall or line of wall elements resisting a common force, or one per 1500 square feet of wall surface, with a minimum of eight tests in any case. The exact test or core location shall be determined at the building site by the licensed engineer or architect responsible for the seismic analysis of the subject building.

In-Place Shear Tests. The bed joints of the outer wythe of the masonry shall be tested in shear by laterally displacing a single brick relative to the adjacent bricks in that wythe. The opposite head joint of the brick to be tested shall be removed and cleaned prior to testing. The minimum quality mortar in 80% of the shear tests shall not be less than the total of 30 psi plus the axial stress in the wall at the point of the test. The shear stress shall be based on the gross area of both bed joints and shall be that at which movement of the brick is first observed.

E.2 Determination of Allowable Stresses in Walls

Design Shear Values. Design seismic in-plane shear stresses shall be substantiated by tests performed as specified in Section E.1.

The allowable shear v_a, shall be 10% of a test value that is determined by the in-plane shear tests. The test value, in psi, is that value that is exceeded by 80% of the test values. The tested shear values are calculated by reducing the test results, in psi, by the axial stress, in psi, in the masonry at the point of testing. The maximum value of v_a is 10 psi. The allowable shear stress, v_a, may be increased by addition of 15% of the axial stress due to the weight of the wall directly above.

Design Compression and Tension Values. Compression stresses for unreinforced masonry having a minimum design shear value of 3 psi shall not exceed 100 psi. Design tension values for unreinforced masonry shall not be permitted.

E.3 Tests of Existing Rod Anchors

Five percent of the existing rod anchors utilized as all or part of the required wall anchors shall be tested in pullout by an approved testing laboratory. The minimum number tested shall be four per floor, with two tests at walls with joists framing into the wall and two tests at walls with joists parallel to the wall. The test apparatus shall be supported on the masonry wall at a minimum distance of the wall thickness from the anchor tested. The rod anchor shall be given a preload of 300 pounds prior to establishing a datum for recording elongation. The tension test load reported shall be recorded at 1/8-inch relative movement of the anchor and the adjacent masonry surface. Results of all tests shall be reported. The report shall include the test results as related to the wall thickness and joist orientation. The allowable resistance value of the existing anchors shall be 40% of the average of those tested anchors having the same wall thickness and joist orientation.

E.4 TESTS FOR WALL ANCHORAGE DEVICES

Qualification tests for devices used for wall anchorage shall be tested with the entire tension load carried on the enlarged head at the exterior face of the wall. Bond on the part of the device between the enlarged head and the interior wall face shall be eliminated for the qualification tests. The resistance value assigned the device shall be 20% of the average of the ultimate loads.

APPENDIX F

MAILING ADDRESSES FOR

REFERENCE STANDARDS LISTED

IN SECTION 4.4.1

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 NCEER 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/1991 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 National Forest Products Association 1250 Connecticut Avenue, N.W. Washington, D.C. 20036 (202) 797-5900 January, 1986

Timber Construction Manual
American Institute of Timber Construction
333 West Hampten Avenue
Englewood, Colorado 80110
(802) 525-1625
1986

Steel: Manual of Steel Construction
American Institute of Steel Construction
400 North Michigan Avenue
Chicago, Illinois 60611
(312) 670-5407
1989 - Allowable Stress Design, 9th 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
1980

Structural Welding Code - AWS D1.1-91
American Welding Society
P.O. Box 351040
Miami, Florida 33135
1988

Standard Specifications, Load Tables and Weight Tables
for Steel Joists and Joist Girders
Steel Joist Institute
1205 - 48th Avenue, North, Suite A
Myrtle Beach, South Carolina 29577
1988

Diaphragm Design Manual
Steel Deck Institute
P.O. Box 9506
Canton, Ohio 44711
(216) 493-7886
1986

Concrete: Building Code Requirements for Reinforced Concrete
ACI 318-89 and ACI 318M-89
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
1988

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 or 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 America (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

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

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LIST OF TECHNICAL REPORTS

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NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341/AS).

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-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-219522/AS).

NCEER-87-0007 "Instantaneous Optimal Control Laws for Tall Buildings Under Seismic Excitations," by J.N. Yang, A. Akbarpour and P. Ghaemmaghami, 6/10/87, (PB88-134333/AS).

NCEER-87-0008 "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures," by Y.J. Park, A.M. Reinhorn and S.K. Kunnath, 7/20/87, (PB88-134325/AS).

NCEER-87-0009 "Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo," by M. Budhu, V. Vijayakumar, R.F. Giese and L. Baumgras, 8/31/87, (PB88-163704/AS). This report is available only through NTIS (see address given above).

NCEER-87-0010 "Vertical and Torsional Vibration of Foundations in Inhomogeneous Media," by A.S. Veletsos and K.W. Dotson, 6/1/87, (PB88-134291/AS).

NCEER-87-0011 "Seismic Probabilistic Risk Assessment and Seismic Margins Studies for Nuclear Power Plants," by Howard H.M. Hwang, 6/15/87, (PB88-134267/AS).

NCEER-87-0012 "Parametric Studies of Frequency Response of Secondary Systems Under Ground-Acceleration Excitations," by Y. Yong and Y.K. Lin, 6/10/87, (PB88-134309/AS).

NCEER-87-0013 "Frequency Response of Secondary Systems Under Seismic Excitation," by J.A. HoLung, J. Cai and Y.K. Lin, 7/31/87, (PB88-134317/AS).

NCEER-87-0014 "Modelling Earthquake Ground Motions in Seismically Active Regions Using Parametric Time Series Methods," by G.W. Ellis and A.S. Cakmak, 8/25/87, (PB88-134283/AS).

NCEER-87-0015 "Detection and Assessment of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 8/25/87, (PB88-163712/AS).

NCEER-87-0016 "Pipeline Experiment at Parkfield, California," by J. Isenberg and E. Richardson, 9/15/87, (PB88-163720/AS). This report is available only through NTIS (see address given above).

NCEER-87-0017 "Digital Simulation of Seismic Ground Motion," by M. Shinozuka, G. Deodatis and T. Harada, 8/31/87, (PB88-155197/AS). This report is available only through NTIS (see address given above).

NCEER-87-0018 "Practical Considerations for Structural Control: System Uncertainty, System Time Delay and Truncation of Small Control Forces," J.N. Yang and A. Akbarpour, 8/10/87, (PB88-163738/AS).

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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-88-0008 "Reliability Analysis of Code-Designed Structures Under Natural Hazards," by H.H-M. Hwang, H. Ushiba and M. Shinozuka, 2/29/88, (PB88-229471/AS).

NCEER-88-0009 "Seismic Fragility Analysis of Shear Wall Structures," by J-W Jaw and H.H-M. Hwang, 4/30/88, (PB89-102867/AS).

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