CHAPTER 6

SEISMIC EVALUATION PROCEDURES FOR HOSPITAL BUILDINGS

ADMINISTRATIVE REGULATIONS FOR THE OFFICE OF STATEWIDE HEALTH PLANNING AND DEVELOPMENT (OSHPD)

ARTICLE 1
DEFINITIONS AND REQUIREMENTS

1.0 Scope. The regulations in this article shall apply to the administrative procedures necessary to implement the seismic retrofit requirements of the Alfred E. Alquist Hospital Facilities Seismic Safety Act of 1983.

1.1 Application. The regulations shall apply to all general acute care hospital facilities as defined in Section 1.2 of these regulations.

1.2 Definitions. Unless otherwise stated, the words and phrases defined in this section shall have the meaning stated therein throughout Chapter 6, Part 1, Title 24.

ALTERNATIVE ANALYSIS means a complete seismic analysis using methodology approved in advance by the Office and meeting the criteria of Article 2, Section 2.7 of these regulations.

BULK MEDICAL GAS SYSTEM means an assembly of fixed equipment such as storage containers, pressure regulators, pressure relief devices, vaporizers, manifolds and interconnecting piping that has a capacity of more than 20,000 cubic feet (NTP) of cryogenic medical gas.

COMMUNICATIONS SYSTEM means the assembly of equipment such as telephone switchgear, computers, batteries, radios, microwave communications systems, towers and antennas that provide essential internal and external communication links.

COMPLETE STRUCTURAL DAMAGE means a significant portion of the structural elements have exceeded their ultimate capacities for some critical structural elements or connections have failed, resulting in dangerous permanent lateral displacement, partial collapse or collapse of the entire building. A Complete Structural Damage would be a loss of 100% of the building's replacement cost.

CONFORMING BUILDING means a building originally constructed in compliance with the requirements of the 1973 or subsequent edition of the *California Building Code.*

CRITICAL CARE AREA means those special care units, intensive care units, coronary care units, angiography laboratories, cardiac catheterization laboratories, delivery rooms, emergency rooms, operating rooms, postoperative recovery rooms and similar areas in which patients are intended to be subjected to invasive procedures and connected to line-operated, electromedical devices.

EMERGENCY POWER SUPPLY (EPS) means the source of electric power including all related electrical and mechanical components of the proper size or capacity, or both, required for the generation of the required electrical power at the EPS output terminals. For rotary energy converters, components of an EPS include the prime mover, cooling system, generator, excitation system, starting system, control system, fuel system and lube system (if required).

ESSENTIAL ELECTRICAL SYSTEMS means a system as defined in the *California Electrical Code,* Article 517 "Health Care Facilities," Chapter 5, Part 3 of Title 24.

FIRE ALARM SYSTEM means a system or portion of a combination system consisting of components and circuits arranged to monitor and annunciate the status of fire alarm or supervisory signal initiating devices and to initiate appropriate response to those signals.

FUNCTIONAL CONTIGUOUS GROUPING means a group of hospital buildings, each of which contains the primary source of one or more basic service that are operationally interconnected in a manner acceptable to the Department of Health Services.

GENERAL ACUTE CARE HOSPITAL as used in Chapter 6, Part 1 means a hospital building as defined in Section 129725 of the Health and Safety Code and that is also licensed pursuant to subdivision (a) of Section 1250 of the Health and Safety Code, but does not include these buildings if the beds licensed pursuant to subdivision (a) of Section 1250 of the Health and Safety Code, as of January 1, 1995, comprise 10 percent or less of the total licensed beds of the total physical plant, and does not include facilities owned or operated, or both, by the Department of Corrections. It also precludes hospital buildings that may be licensed under the above mentioned code sections, but provide skilled nursing or acute psychiatric services only.

HOSPITAL EQUIPMENT means equipment permanently attached to the building utility services such as surgical, morgue, and recovery room fixtures, radiology equipment, medical gas containers, food service fixtures, essential laboratory equipment, TV supports, etc.

HYBRID STRUCTURE means a structure consisting of an original and one or more additions, constructed at different times, and with lateral-force-resisting systems of different types, or constructed with differing materials or a different design approach. The original building and additions are interconnected and not seismically isolated.

NONCONFORMING BUILDING means any building that is not a conforming building.

NONSTRUCTURAL PERFORMANCE CATEGORY (NPC) means a measure of the probable seismic performance of building contents and nonstructural systems critical to providing basic services to inpatients and the public following an earthquake, as defined in Article 11, Table 11.1 of these regulations.

PRIMARY SOURCE means that building or portion of a building identified by the hospital as housing the main or principal source of a basic hospital service, serving the greatest number of patients, providing the greatest number of patient beds, or having the largest/greatest floor space of the specified basic service. The hospital may submit data to substantiate the primary source through alternative criteria if different than above.

PRINCIPAL HORIZONTAL DIRECTIONS means the two predominant orthogonal translational modes of vibration with the lowest frequency.

PROBABILITY OF COLLAPSE means the fraction of building that is expected to collapse given that the ground motions defined in Section 1.4.5.1.2.1.4 occur at the building site.

SIGNIFICANT STRUCTURAL DEFICIENCY means an attribute of the structure considered to be significant with respect to Probability of Collapse.

SLENDER SEISMIC RESISTING SYSTEM means any vertical system for resisting lateral forces, such as walls, braced frames or moment frames, with a height to width ratio greater than four for the minimum horizontal dimension at any height.

STRUCTURAL PERFORMANCE CATEGORY (SPC) means a measure of the probable seismic performance of building structural systems and risk to life posed by a building subject to an earthquake, as defined in Article 2, Table 2.5.3 of these regulations.

1.3 Seismic evaluation. All general acute care hospital owners shall perform a seismic evaluation on each hospital building in accordance with the Seismic Evaluation Procedures as specified in Articles 2 through 11 of these regulations. By January 1, 2001, hospital owners shall submit the results of the seismic evaluation to the Office for review and approval. By completing this seismic evaluation, a hospital facility can determine its respective seismic performance categories for both the Structural Performance Category (SPC) and the Nonstructural Performance Category (NPC) in accordance with Articles 2 and 11 of these regulations.

1.3.1 Seismic evaluation submittal. Hospital owners shall submit the seismic evaluation report to the Office by January 1, 2001. There are no provisions for submittal of the evaluation report after this date, except as provided in Section 1.4.5.1.2. The hospital owners shall submit the evaluation report in accordance with Section 7-113, "Application for Plan Report or Seismic Compliance Extension Review" and Section 7-133, "Fees" of Article 3, Chapter 7, Part 1, Title 24.

Exceptions:

1. Any hospital facility owner whose building is exempted from the structural evaluation per Section 2.0.1.2 shall not be required to submit a structural evaluation report as specified in Section 1.3.3. In lieu of the structural evaluation report, hospital owners shall submit the matrix of construction information for the specified building(s) as noted in Section 1.3.4.6 to the Office by January 1, 2001;
2. Any hospital facility owner whose building is exempted from the nonstructural seismic evaluation per Section 11.0.1.2 shall not be required to submit a nonstructural evaluation report as specified in Section 1.3.4. In lieu of the nonstructural evaluation report, hospital owners shall submit the matrix of construction information for the specified building(s) as noted in Section 1.3.4.6 to the Office by January 1, 2001.

1.3.2 Seismic evaluation format. The evaluation shall consist of the Structural Evaluation and the Nonstructural Evaluation Reports. The reports shall be prepared in conformance with Part 1, Chapter 7, Title 24 and these regulations and prepared as follows:

1. Reports shall be submitted in an $8^1/_2''$ x 11″ format;
2. All site, architectural, and engineering plans shall be formatted on 11- by 17-inch sheets (folded to $8^1/_2$ by 11 inches);
3. Larger sheets, if required to clearly describe the requested information, shall be appended to the reports; and
4. Other supporting documents in addition to those meeting the minimum requirements of Sections 1.3.3 and 1.3.4 may be appended to the reports.

1.3.3 Structural evaluation report. The structural evaluation report shall include the following elements:

1. A description of the building, including photographs of the building, and sketches of the lateral force resisting system;
2. The "General Sets of Evaluation Statements" from the Appendix;
3. A synopsis of the investigation and supporting calculations that were made;
4. A list of the deficiencies requiring remediation to change statement responses from false to true; and
5. The SPC for the building, with comments on the relative importance of the deficiencies.

1.3.4 Nonstructural evaluation report. The nonstructural evaluation report shall include the following elements:

1. A written description of the evaluation methods and procedures conducted in conformance with Article 11 of these regulations for the determination of the facilities existing compliance. The description shall include the systems and components required for the planned level of nonstructural performance as identified in Table 11.1;

   Exceptions:

   1. Additional evaluations as per Section 11.01.3 will be required for any hospital owner electing to obtain a higher NPC at a future date consistent with an approved compliance plan;
   2. A complete nonstructural evaluation up to NPC 5 is required prior to the hospital owner selling or leasing the hospital to another party.

2. Provide single line diagrammatic plans (site plan and floor plans) of the following:

   2.1 Location of the following areas/spaces:

   (a) Central supply areas;

   (b) Clinical laboratory service spaces;

   (c) Critical care areas;

(d) Pharmaceutical service spaces;

(e) Radiological service spaces; and

(f) Sterile supply areas.

2.2 Diagrammatic or narrative descriptions of the following major building systems where deficiencies are identified that are within the scope of the evaluation, including primary source location or point(s) of entry into the building and major distribution routes of each utility or system.

(a) Mechanical systems including:

i. Air supply equipment, piping, controls and ducting;

ii. Air exhaust equipment and ducting;

iii. Steam and hot water piping systems, including boilers, piping systems, valving and components; and

iv. Elevators selected to provide service to patient, surgical, obstetrical and ground floors.

(b) Plumbing systems including:

i. Domestic water supply system, including heating equipment, valving, storage facilities and piping;

ii. Medical gas supply system, including storage facilities, manifolding and piping;

iii. Fire protection system, including sprinkler systems, wet and dry standpipes, piping systems and other fire suppression systems; and

iv. Sanitary drainage system, including storage facilities and piping.

(c) Electrical systems, including:

i. Essential electrical system, including emergency fuel storage;

ii. Internal communication systems;

iii. External communication systems;

iv. Fire alarm systems; and

v. Elevators selected to provide service to patient, surgical, obstetrical and ground floors.

Building name/ designation	OSHPD (or local building) permit date/ number	Governing building code	Construction completion date	Building type (per Section 2.2.3)	SPC	NPC

3. A synopsis of the evaluation and all the calculations used in the course of the evaluation for the planned level of nonstructural performance;
4. A list of the deficiencies identified in the course of the evaluation for the planned level of nonstructural performance;
5. Provide an 11- by 17-inch scaled Site Plan which identifies the boundaries of the facility property, locates all buildings, roadways, parking and other significant site features and improvements. Identify boundaries between buildings which were constructed at different times. For all buildings, note the names of the buildings and date of each related building permit. Provide the SPC and NPC for all buildings.
6. Provide the following matrix of construction information for each building of the facility under the acute care license, include the Structural Performance Category (SPC) and Nonstructural Performance Category (NPC) for all hospital buildings (see Tables 2.5.3 and 11.1). Identify each building addition separately. For buildings constructed, reconstructed or remodeled under a building permit issued by the Office, provide the OSHPD application number and the date of the initial submittal.

1.4 Compliance plans. A compliance plan shall be prepared and submitted for each building subject to these regulations. All general acute care hospital owners shall formulate a compliance plan which shall indicate the facilities intent to do any of the following:

1. Building retrofit for compliance with these regulations for continued acute care operation beyond 2030;
2. Partial retrofit for initial compliance, with closure or replacement expected by 2002, 2008, 2013 or 2030;
3. Removal from acute care service with conversion to nonacute care health facility use; or
4. No action, building to be closed, demolished or replaced.

This plan must clearly indicate the actions to be taken by the facility and must be in accordance with the timeframes set forth in Article 2 (Structural Performance Category-“SPC”) and Article 11 (Nonstructural Performance Category-“NPC”) of the Seismic Evaluation Procedure regulations.

1.4.1 Preparation of the compliance plan. The Compliance Plan shall be prepared and submitted in conformance with these regulations in the following format:

1. Compliance Plans shall be submitted in an $8^1/_2$- by 11-inch format;
2. All site, architectural, and engineering plans shall be formatted on 11- by 17-inch sheets (folded to $8^1/_2$ by 11 inches);
3. Larger sheets, if required to clearly describe the requested information, shall be appended to the compliance plan; and
4. Other supporting documents in addition to those meeting the minimum requirements of Section 1.4.4 may be appended to the compliance plan.

1.4.2 Compliance plan submittal. Hospital owners shall submit the compliance plan to the Office by January 1, 2001, unless the owner requests an extension pursuant to Section 1.4.3. The hospital owners shall submit the compliance plan in accordance with Section 7-113, "Application for Plan or Report Review" and Section 7-133, "Fees" of Article 3, Chapter 7, Part 1, Title 24.

1.4.3 Compliance plan submittal extension. Hospital owners may request an extension from the Office for submission of the compliance plan. Any hospital owner requesting an extension for submittal of the compliance plan shall make such request in writing to the Office up to 180 days prior to, but no later than January 1, 2001. The compliance plan must be submitted no later than January 1, 2002. All hospital owners requesting an extension for submittal of the compliance plan shall certify to OSHPD that all hospital buildings continuing acute care operation beyond January 1, 2002 meet the standards of NPC 2 by January 1, 2002.

1.4.4 Compliance plan requirements. Each compliance plan shall contain the following elements:

1. An Existing Site/Campus Description;
2. A Compliance Plan Description;
3. A Compliance Site Plan;
4. A Compliance Plan Schedule; and
5. An Existing and Planned Buildings Matrix.

1.4.4.1 Existing site/campus description. If the compliance plan is submitted separately from the seismic evaluation, it will be necessary to resubmit the information as specified in Section 1.3.4.5, of the Nonstructural Evaluation Report.

1.4.4.2 Compliance plan description. Provide a comprehensive narrative description of the Compliance Plan, including the projected schedule for compliance.

1.4.4.3 Compliance site plan. Provide Compliance Site Plans, indicating the configuration of the facility at the 2008 and 2030 milestones. The plans shall indicate conforming and nonconforming buildings and identify the final configuration of the facility at each milestone, after completion of compliance measures.

1.4.4.4 Compliance plan schedule. Provide a bar graph schedule which describes the schedule for compliance with the SPC and NPC seismic performance categories, indicating the schedule of the following major phases of the plan:

1. Obtain a geotechnical report (if necessary);
2. Architecture and engineering design/construction document preparation;
3. Local approvals;
4. Office review, approval and permitting;
5. Approval of Department of Health Services Licensing and Certification, and any other required licensing;
6. Permanent relocation of acute care services to other buildings or facilities (identify services affected);
7. Temporary/interim relocation of acute care services to other buildings including the duration of the approved program flexibility plan pursuant to Health and Safety Code Section 1276.05;
8. Construction period; and
9. Beneficial occupancy.

1.4.4.5 Existing and planned buildings matrix. Provide the following matrix of construction information for each building of the facility under the acute care license, include the Structural Performance Category (SPC) and Nonstructural Performance Category (NPC) for all hospital buildings (see Tables 2.5.3 and 11.1). Identify each building addition separately.

Building name/ designation	Building type (per Section 2.2.3)	SPC existing	SPC planned	NPC existing	NPC planned

1.4.5 Compliance plan update/change notification. Should a hospital owner change an approved Compliance Plan, the hospital shall document any changes and submit for review and approval to the Office an amended Compliance Plan. Changes are defined as alterations to the planned level of seismic performance or compliance schedule. Submittal of an amended compliance plan shall require a hospital owner to comply with one or more of the following provisions, if applicable:

1. A hospital owner shall submit to the Department of Health Services' Seismic Safety Unit (DHS) an Office-approved compliance plan that includes interim relocation of general acute care services in accordance with a program flexibility plan pursuant to Health and Safety Code Section 1276.05. This submittal by the hospital owner to DHS shall occur within 30 days of the Office's approval.
2. A hospital owner shall comply with the requirements of Section 1.5.2, "Delay in Compliance" for any amended compliance plan.
3. A hospital owner amending a compliance plan to attain a higher NPC level will perform a nonstructural evaluation of the systems and components required for the planned level of nonstructural performance identified in Table 11.1, "Nonstructural Performance Categories."

1.4.5.1 Change in seismic performance category. The SPC or NPC for a hospital building may be changed by the Office from the initial determination in Section 1.3.3 or 1.3.4, provided the building has been modified to comply with the requirements of Chapter 34A, Part 2 of Title 24 for the specified SPC or NPC. The SPC of a hospital building may also be changed by the Office on the basis of collapse probability assessments in accordance with Section 1.4.5.1.2.

1.4.5.1.1 The SPC or NPC for a hospital building may be changed by the Office from the initial determination made per Sections 2.0.1.2.3 or 11.0.1.2.1 upon the following:

1. A Seismic Evaluation Report shall be submitted and approved which shall include either or both of the following:

   1.1 A structural evaluation report in accordance with Section 1.3.3;

1.2 A nonstructural evaluation report in accordance with Section 1.3.4.

Exception: To change an NPC 1 hospital building to an NPC 2 under this section, the nonstructural evaluation may be limited in scope to the systems and equipment specified in Section 11.2.1.

2. The building has been modified to comply with the requirements of Chapter 34A, Part 2 of Title 24 for the specified SPC or NPC.

1.4.5.1.2 Hospital buildings with an SPC 1 rating, may be reclassified to SPC 2 by the Office, pursuant to Table 2.5.3, on the basis of a collapse probability assessment, provided the hospital buildings received an extension to the January 1, 2008, compliance deadline in accordance with Section 1.5.2.

Exception: Hospital buildings with the following deficiencies are not eligible for reclassification:

a) The potential for surface fault rupture and surface displacement at the building site is present (Section 9.3.3).

b) Buildings with unreinforced masonry bearing wall construction (Section 5.4).

1.4.5.1.2.1 The collapse probability assessment by the Office shall be determined using the following:

1. Multi-Hazard Loss Estimation Methodology, Earthquake Module (HAZUS-MH MR 2) developed by the Federal Emergency Management Agency (FEMA) / National Institute of Building Sciences (NIBS).
2. Building specific input parameters required by the Advanced Engineering Building Module (AEBM) of the HAZUS methodology shall be obtained from Appendix H to Chapter 6.
3. Modifications by the Office to the AEBM input parameters are hereby adopted as shown in Appendix H to Chapter 6, which are based on the following:

   a) Building type

   b) Building height and number of stories

   c) Building age

   d) Significant Structural Deficiencies listed in Section 1.4.5.1.2.2.2.2.

4. Site seismicity parameters adjusted for soil type, as determined by the Office, shall be the lesser of:

   a) Deterministic ground motion due to the maximum magnitude earthquake event on the controlling fault system.

   b) Probabilistic ground motion having 10% probability of being exceeded in 50 years.

1.4.5.1.2.2 Hospital buildings with SPC 1 rating may be reclassified as follows:

1. The Office shall issue a written notice to the hospital owners informing them that they may be eligible for reclassification of their SPC 1 buildings as permitted by Section 1.4.5.1.2.
2. For a building to be considered for reclassification, the hospital owner shall submit the following by July 1, 2009:

2.1 A complete seismic evaluation of the building pursuant to Section 1.3.3.

Exception: Hospital owners who had submitted a complete structural evaluation report in compliance with Section 1.3.3, that is deemed to be complete by the Office, need not resubmit.

2.2 A supplemental evaluation report prepared by a California registered structural engineer that identifies the existence or absence of the building structural Lateral Force Resisting System (LFRS) properties and Significant Structural Deficiencies listed below:

a. Age: Year of the *California Building Code* (CBC) used for the original building design.

   Exception: For pre-1933 buildings, the design year shall be reported.

b. Materials Tests: Office approved materials test results based on test plan preapproved by the Office (Section 2.1.2).

c. Mass irregularity (Section 3.3.4).

d. Vertical discontinuity (Section 3.3.5).

e. Short captive column (Section 3.6).

f. Material deterioration (Section 3.7).

g. Weak columns (Sections 4.2.8 and 4.3.6).

h. Wall anchorage (Section 8.2).

i. Redundancy (Section 3.2).

j. Weak story irregularity (Section 3.3.1).

k. Soft story irregularity (Section 3.3.2).

l. Torsional irregularity (Section 3.3.6).

m. Deflection incompatibility (Section 3.5).

n. Cripple walls (Section 5.6.4).

o. Topping slab missing (Sections 7.3 and 7.4) or the building type (structural system) is of lift slab construction.

This supplemental evaluation report shall include supporting documentation relating to the existence or absence of the Significant Structural Deficiencies listed above including calculations, where required, for review and acceptance by the Office, unless they are included in the complete structural evaluation.

2.3 Building systems shall be classified as to their Model Building Type per Table 1.4.5.1. For buildings with multiple building types, all types shall be listed. The building type resulting in the maximum collapse probability will be utilized by the Office to determine eligibility for reclassification.

TABLE 1.4.5.1—MODEL BUILDING TYPE

MODEL BUILDING TYPE (MBT)	DESCRIPTION
W1	Wood, Light Frame (≤ 5,000 sq ft)
W2	Wood, greater than 5,000 sq ft
S1	Steel Moment Frame
S2	Steel Braced Frame
S3	Steel Light Frame
S4	Steel Frame with Cast-In Place Concrete Shear Walls
S5	Steel Frame with Unreinforced Masonry Infill Walls
C1	Concrete Moment Frame
C2	Concrete Shear Walls
C3	Concrete Frame with Unreinforced Masonry Infill Walls
PC1	Precast Concrete Tilt-Up Walls
PC2	Precast Concrete Frames with Concrete Shear Walls
RM1	Reinforced-masonry Bearing Walls with Wood or Metal Deck Diaphragms
RM2	Reinforced-masonry Bearing Walls with Concrete Diaphragms
URM	Unreinforced-masonry Bearing Walls
MH	Manufactured Housing

2.4 Building height and number of stories above and below the seismic base shall be specified.

1.4.5.1.2.3 Upon assessment of the collapse probability of the SPC-1 building, the Office shall notify the hospital owner in writing the final SPC rating of the subject building.

1.4.5.1.2.4 When the collapse probability assessment by the Office results in the building remaining in SPC 1, further evaluation may be provided by the hospital owner in accordance with Section 2.7 in order to substantiate a higher SPC rating.

1.4.5.1.3 Except as provided in Section 1.4.5.1.4, a nonconforming hospital building that does not meet the structural and nonstructural requirements of Table 2.5.3 and Table 11-1 shall not provide acute care services or beds after the compliance deadlines set forth in Section 1.5.1. After these deadlines, the following shall apply.

1. A nonconforming hospital building used as a hospital outpatient clinical services building shall not be classified as a hospital building. It shall comply with the provisions of Health and Safety Code Section 129725. It shall not be subject to the requirements of Title 24, Part 1, Chapter 6.
2. A nonconforming hospital building used as an acute psychiatric hospital or multistory skilled nursing facility or intermediate care facility shall be classified as a hospital building. However, it shall not be subject to the requirements of Title 24, Part 1, Chapter 6.
3. A nonconforming hospital building used as a single-story wood frame or light steel frame skilled nursing facility or intermediate care facility shall not be classified as a hospital building, and shall not be subject to the requirements of Title 24, Part 1, Chapter 6.
4. A nonconforming hospital building used for purposes other than those listed above shall not be classified as a hospital building; shall not be licensed pursuant to Health and Safety Code Section 1250(a); shall not be subject to the requirements of Title 24, Part 1, Chapter 6; and shall not be under the jurisdiction of the Office.

1.4.5.1.4 A hospital building from which acute care services and beds have been removed shall not provide such services unless it has been modified to comply with the requirements of SPC 5 and NPC 4 or 5. Prior to use for acute care service, the SPC and/or NPC of the hospital building shall be changed in accordance with Section 1.4.5.1.1.

1.5 Compliance requirements. All general acute care hospital owners shall comply with the seismic performance categories, both SPCs and NPCs, established in the seismic evaluation procedures, Articles 2 and 11 and set forth in Tables 2.5.3 and 11.1, respectively.

1.5.1 Compliance deadlines.

1. After January 1, 2002, any general acute care hospital building which continues acute care operation must, at a minimum, meet the nonstructural requirements of NPC 2, as defined in Article 11, Table 11.1 or shall no longer provide acute care services.
2. After January 1, 2008, any general acute care hospital building which continues acute care operation must, at a minimum, meet the structural requirements of SPC 2, as defined in Article 2, Table 2.5.3 or shall no longer provide acute care services.

    Exception: A general acute care hospital may request a delay of SPC 2 requirements if the conditions of Section 1.5.2 are met.

3. After January 1, 2008, any general acute care hospital which continues acute care operation must, at a minimum, meet the nonstructural requirements of NPC 3, as defined in Article 11, Table 11.1 or shall no longer provide acute care services.

   Exception: A general acute care hospital may request an exemption from the anchorage and bracing requirements of NPC 3 if all the conditions of Section 1.5.2, Item 2, are met.

4. After January 1, 2030, any general acute care hospital building which continues acute care operation must, at a minimum, meet the structural requirements of SPC 3, 4 or 5, as defined in Article 2, Table 2.5.3 and the nonstructural requirements of NPC 5, as defined in Article 11, Table 11.1 or shall no longer provide acute care services.

1.5.2 Delay in compliance.

1. The Office may grant the hospital owner an extension to the January 1, 2008 seismic compliance deadline for both structural and nonstructural requirements if compliance will result in diminished health care capacity which cannot be provided by other general acute care hospitals within a reasonable proximity.

   1.1 Hospital owners requesting an extension in accordance with Section 1.5.2 must submit an application form to the Office by January 1, 2007. The application form shall be accompanied by a statement explaining why the hospital is seeking the extension to the January 1, 2008 seismic compliance deadline. The statement shall include, at a minimum, the following information:

   (a) The length/duration of the extension request;

   (b) The hospital buildings requiring an extension; and

   (c) The acute care services that will be completely or partially unavailable if the extension is denied.

   1.2 The hospital owner shall request an extension for seismic compliance in one year increments, up to a maximum of five years, beyond the mandated year of compliance. The hospital owner shall also submit an amended compliance plan and schedule in accordance with Section 1.4.5 indicating when compliance will be obtained.

2. Any general acute care hospital located in Seismic Zone 3, as defined by Section 1627B.2 of the 1998 *California Building Code,* may request an exemption from the anchorage and bracing requirements of NPC 3 if all the following conditions are met:

   2.1 The hospital must meet the anchorage and bracing requirements for NPC 2 by January 1, 2002;

   2.2 The hospital shall submit a site-specific engineering geologic report, prepared in accordance with Section 1634A.1 of the 1995 *California Building Code.* The report shall include estimates of the effective peak ground acceleration (EPA) with a 10 percent probability of exceedance in 50 years;

   2.3 The California Geological Survey (CGS) reviews and approves the findings of the site-specific engineering geologic report;

   2.4 The site-specific engineering geologic report demonstrates that the estimated EPA with a 10 percent probability of exceedance in 50 years is less than 0.25 g;

   2.5 The hospital owner requesting the exemption shall pay the actual costs of OSHPD and CGS for the review and approval of the site-specific engineering geologic report.

3. Any SPC-1 building which is part of the functional contiguous grouping of a general acute care hospital may receive a five-year extension to the January 1, 2008 deadline for both structural and nonstructural requirements under the following conditions:

   3.1 The owner must apply for an extension with the Office no later than January 1, 2004;

   3.2 The owner must submit an amended compliance plan to the Office by July 1, 2004;

   3.3 The buildings must have met the NPC-2 nonstructural requirements by January 1, 2002;

   3.4 At least one building within the contiguous grouping shall have obtained a building permit prior to 1973 and shall have been evaluated and classified as SPC-1 in accordance with Section 1.3;

   Exception: Hospital buildings that were classified as SPC-1 under Section 2.0.1.2.3 must submit a structural evaluation report in accordance with Sections 1.3.2 and 1.3.3 by January 1, 2004.

   3.5 The basic service(s) from the building shall be:

   (a) Relocated to an SPC-3, 4, or 5/NPC-4 or 5 building by January 1, 2013.

      i. The building shall not be used for general acute care service after January 1, 2013, unless it has been retrofitted to an SPC-5/NPC-4 or 5 building; or

   (b) Continued in building if it is retrofitted to an SPC-5/NPC-4 or 5 building by January 1, 2013;

   3.6 Any other SPC-1 building in the contiguous grouping other than the building identified in subsection 1.5.2.3.4 must be retrofitted to at least an SPC-2/NPC-3 by January 1, 2013, or no longer used for acute care hospital inpatient services.

4. A post-1973 building classified as SPC-3 or 4 may receive an extension to the January 1, 2008, deadline for both the structural and nonstructural requirements, provided it will be closed to general acute care inpatient service by January 1, 2013. The basic services in this building shall be relocated to an SPC-5/NPC-4 or 5 building by January 1, 2013;

4.1 Any SPC-1 building in a functional contiguous grouping must be retrofitted to at least an SPC-2/NPC-3 by January 1, 2013, or no longer used for acute care hospital inpatient services. The following conditions apply to these hospital buildings:

(a) The owner must apply for an extension with the Office no later than January 1, 2004;

(b) The owner must submit an amended compliance plan to the Office by July 1, 2004; and

(c) The buildings must have met the NPC-2 nonstructural requirements by January 1, 2002.

5. A single building containing all of the basic services may receive a five-year extension to the January 1, 2008, deadline for both structural and nonstructural requirements under the following conditions:

5.1 The owner must apply for an extension with the Office no later than January 1, 2004;

5.2 The owner must submit an amended compliance plan to the Office by July 1, 2004;

5.3 The building shall have obtained a building permit prior to 1973 and shall have been evaluated and classified as SPC-1 in accordance with Section 1.3;

Exception: Hospital buildings that were classified as SPC-1 under Section 2.0.1.2.3 must submit a structural evaluation report in accordance with Sections 1.3.2 and 1.3.3 by January 1, 2004.

5.4 The basic services from this building shall be:

(a) Relocated to an SPC-3, 4, or 5/NPC-4 or 5 building by January 1, 2013.

i. The building shall not be used for general acute care service after January 1, 2013, unless it has been retrofitted to an SPC-5/NPC-4 or 5 building; or

(b) Continued in building if it is retrofitted to an SPC-5/NPC-4 or 5 building by January 1, 2013.

1.6 Dispute resolution/appeals process. Dispute resolution and appeals shall be in conformance with Article 5, Chapter 7, Part 1 of Title 24.

1.7 Notification from OSHPD.

1. The Office shall issue written notices of compliance to all hospital owners that have attained the minimum required SPC and NPC performance levels by January 1, 2008, January 1, 2013, and January 1, 2030;
2. The Office shall issue written notices of violation to all hospital owners that are not in compliance with the minimum SPC and NPC performance levels by January 1, 2008, January 1, 2013, and January 1, 2030; and
3. The Office shall notify the State Department of Health Services of the hospital owners which have received a written notice of violation for failure to comply with these regulations.

ARTICLE 2
PROCEDURES FOR STRUCTURAL EVALUATION OF BUILDINGS

2.0 General.

2.0.1 Structural evaluation procedure.

1. The structural evaluation process shall include the following steps:

1.1 Site visit and data collection;

1.2 Identification of building type;

1.3 Completion of evaluation statements in appendix;

1.4 Follow-up field work, if required;

1.5 Follow-up analysis for “False” evaluation statements;

1.6 Final evaluation for the building;

1.7 Preparation of the evaluation report; and

1.8 Submittal of evaluation report to OSHPD.

2. A general acute care hospital facility building may be exempted from a structural evaluation upon submittal of a written statement by the hospital owner to OSHPD certifying the following conditions:

2.1 A conforming building as defined in Article 1, Section 1.2, may be placed into SPC 5 in accordance with Table 2.5.3 under the following circumstances:

(a) The building was designed and constructed to the 1989 or later edition of Part 2, Title 24, and

(b) If any portion of the structure, except for the penthouse, is of steel moment resisting frame construction (Building Type 3, or Building Type 4 or 6 with dual lateral system, as defined in Section 2.2.3) and the building permit was issued after October 25, 1994.

2.2 All other conforming buildings as defined in Article 1, Section 1.2, may be placed into SPC 4 in accordance with Table 2.5.3, except those required by Section 4.2.10 to be placed in SPC 3 in accordance with Table 2.5.3, without the need for any structural evaluation.

2.3 Nonconforming buildings as defined in Article 1, Section 1.2 may be placed into SPC 1 in accordance with Table 2.5.3 without any structural evaluation.

2.1 Site visit, evaluation and data collection procedures.

2.1.1 Site visit and evaluation.

1. The evaluator shall visit the building to observe and record the type, nature and physical condition of the structure.

2. The evaluator shall review an *Engineering Geological Report* on site geologic and seismic conditions. The report shall be prepared in accordance with Title 24, Section 1634A of 1995 *California Building Code* (CBC) or equivalent provision in later version of the CBC.

   Exceptions:

   1. Reports are not required for one-story, wood-frame and light steel-frame buildings of Type II or Type V construction and 4,000 square feet or less in floor area;
   2. A previous report for a specific site may be resubmitted, provided that a reevaluation is made and the report is found by the Office to be currently appropriate.

3. Establish the following *site and soil parameters:*

   a. The value of the effective peak acceleration coefficient (A_a) from Figure 2.1 and 2.1a;

   b. The value of the effective peak velocity-related acceleration coefficient (A_v) from Figure 2.1 and 2.1a;

   c. The soil profile type (S_1, S_2, S_3 or S_4) derived from the geotechnical report or from Table 2.1;

   d. The site coefficient, (S), from Table 2.1; and

   e. The ground motion parameters and near field effects in strong ground shaking required for the evaluation of welded steel moment frame structures per Sections 4.2.0.1, 4.2.0.2 and 4.2.10.

4. Assemble building design data including:

   a. Construction drawings, specifications and calculations for the original building (Note: when reviewing and making use of existing analyses and structural member checks, the evaluator shall assess and report the basis of the earlier work);

   b. All drawings, specifications and calculations for remodeling work; and

   c. Material tests and inspection reports for nonconforming buildings. If the original drawings are available, but material test and inspection reports are not available, perform the testing program as specified in Section 2.1.2.2.

   If structural drawings are not available, the site visit and evaluation shall be performed as described in Section 2.1.1.5, and structural data shall be collected using the procedures in Sections 2.1.2.1 and 2.1.2.2.

5. During the site visit, the evaluator shall:

   a. Verify existing data;

   b. Develop other needed data (e.g., measure and sketch building as outlined in Section 2.1.2);

   c. Verify the vertical and lateral systems;

   d. Check the condition of the building; and

   e. Identify special conditions, anomalies and oddities.

6. Review other data available such as assessments of building performance following past earthquakes.
7. Prepare a summary of the data using an OSHPD-approved format.
8. Perform the evaluation using the procedures in Sections 2.2 through 2.5.
9. Prepare a report of the findings of the evaluation using an OSHPD-approved format.

2.1.2 Data collection. Building information pertinent to a structure's seismic performance, including condition, configuration, detailing, material strengths and foundation type, shall be obtained in accordance with this section, and documented on drawings and/or sketches that shall be included with the structural calculations.

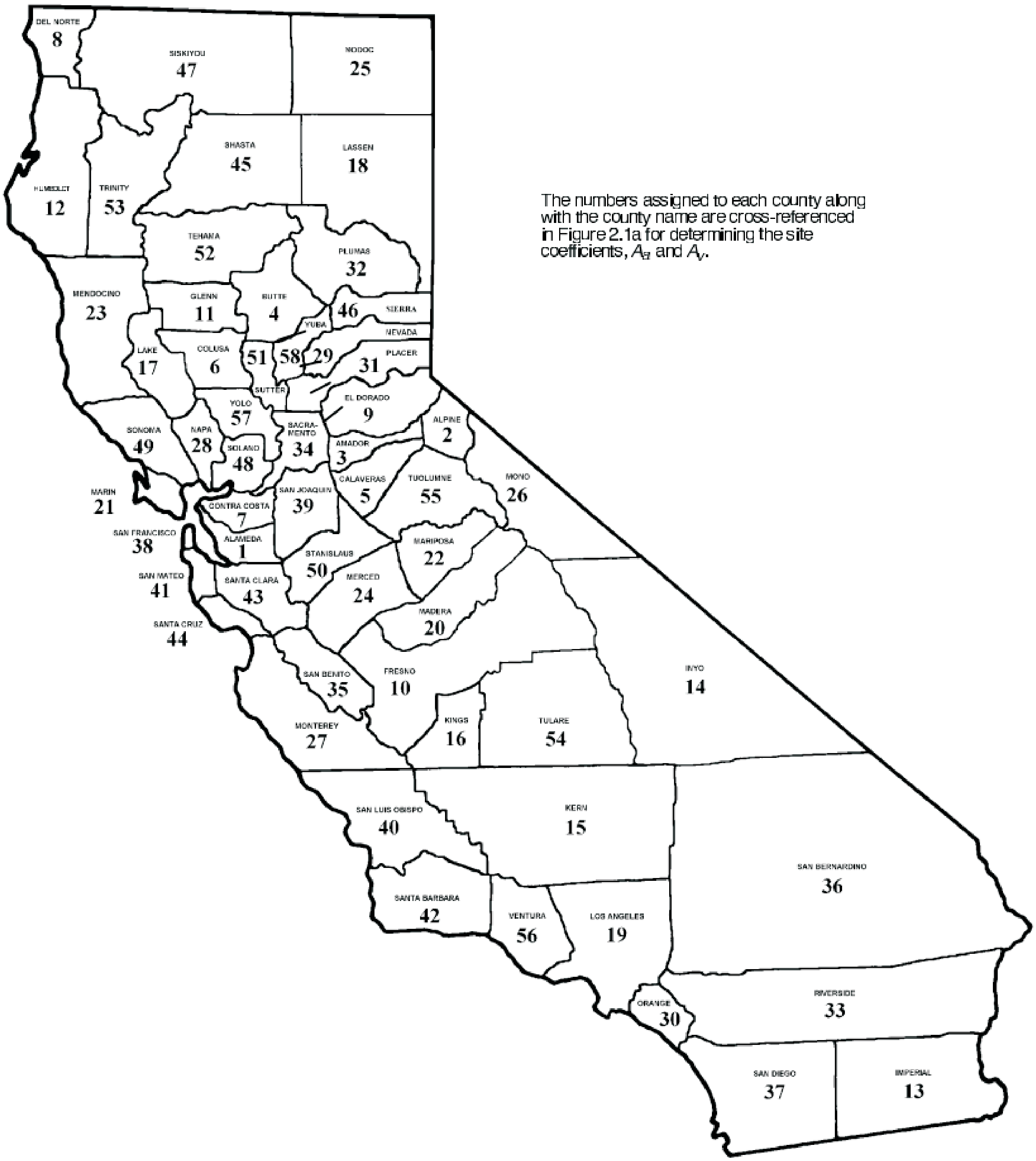
Exception: Materials testing is not required for reclassification by the collapse probability assessment option as permitted by Section 1.4.5.1.2, where nonavailability of materials test is identified as a deficiency per Section 1.4.5.1.2.2.2.2 (b).

2.1.2.1 Building characteristics. Characteristics of the building relevant to its seismic performance shall be obtained for use in the building evaluation. This shall include current information on the building's condition, configuration, material strengths, detailing and foundation type. This data shall be obtained from:

1. Review of construction documents;
2. Destructive and nondestructive testing and examination of selected building components; and
3. Field observation of exposed conditions.

The characteristics of the building shall be established, including identification of the gravity- and lateral-load-carrying systems. The effective lateral-load carrying system may include structural and nonstructural elements that will participate in providing lateral resistance, although these elements may not have intended to provide lateral resistance. The load path shall be identified, taking into account the effects of any modifications, alterations or additions.

2.1.2.1.1 Nonconforming buildings without construction documents. Where the available construction documents do not provide sufficient detail to characterize the structure, the evaluation may be based on field surveys, summarized in as-built drawings. These drawings must depict building dimensions, component sizes, reinforcing information (for concrete and masonry elements), connection details, footing information, and the proximity of neighboring structures. All parts of the building that may contribute to the seismic resistance or that may be affected by the seismic response of the structure must be identified. The field survey shall establish the physical existence of the structural members, and identify critical load bearing members, transfer mechanisms, and connections. The survey shall include information on the structural elements and connector materials and details. Performing the field survey will entail removal of fireproofing or concrete encasement at critical locations to permit direct visual inspection and measurement of elements and connections. Nondestructive techniques such as radiographic, electromagnetic and other methods may be used to supplement destructive techniques.



The numbers assigned to each county along with the county name are cross-referenced in Figure 2.1a for determining the site coefficients, A_a and A_v.

FIGURE 2.1

FIGURE 2.1a—EFFECTIVE PEAK ACCELERATION COEFFICIENT (A_a) AND EFFECTIVE PEAK VELOCITY COEFFICIENT (A_v) FOR CALIFORNIA

No.	County	EPA A_a	EPV A_v		No.	County	EPA A_a	EPV A_v
1	Alameda	0.40	0.40		30	Orange	0.40	0.40
2	Alpine	0.20	0.20		31	Placer	0.20	0.20
3	Amador	0.20	0.20		32	Plumas	0.20	0.20
4	Butte	0.20	0.20		33	Riverside	0.40	0.40
5	Calaveras	0.20	0.20		34	Sacramento	0.20	0.30
6	Colusa	0.20	0.30		35	San Benito	0.40	0.40
7	Contra Costa	0.40	0.40		36	San Bernardino	0.40	0.40
8	Del Norte	0.20	0.20		37	San Diego	0.40	0.40
9	El Dorado	0.20	0.20		38	San Francisco	0.40	0.40
10	Fresno	0.40	0.40		39	San Joaquin	0.30	0.30
11	Glenn	0.20	0.20		40	San Luis Obispo	0.40	0.40
12	Humboldt	0.20	0.30		41	San Mateo	0.40	0.40
13	Imperial	0.40	0.40		42	Santa Barbara	0.40	0.40
14	Inyo	0.40	0.40		43	Santa Clara	0.40	0.40
15	Kern	0.40	0.40		44	Santa Cruz	0.40	0.40
16	Kings	0.40	0.40		45	Shasta	0.20	0.20
17	Lake	0.30	0.30		46	Sierra	0.20	0.20
18	Lassen	0.20	0.20		47	Siskiyou	0.20	0.20
19	Los Angeles	0.40	0.40		48	Solano	0.40	0.40
20	Madera	0.20	0.30		49	Sonoma	0.40	0.40
21	Marin	0.40	0.40		50	Stanislaus	0.40	0.40
22	Mariposa	0.20	0.30		51	Sutter	0.20	0.20
23	Mendocino	0.40	0.40		52	Tehama	0.20	0.20
24	Merced	0.40	0.40		53	Trinity	0.20	0.30
25	Modoc	0.20	0.20		54	Tulare	0.40	0.40
26	Mono	0.40	0.40		55	Tuolumne	0.20	0.20
27	Monterey	0.40	0.40		56	Ventura	0.40	0.40
28	Napa	0.40	0.40		57	Yolo	0.20	0.30
29	Nevada	0.20	0.20		58	Yuba	0.20	0.20

TABLE 2.1—SOIL PROFILE TYPES AND SITE COEFFICIENTS

SOIL PROFILE TYPE	PROFILE WITH	SITE COEFFICIENT, *S*
*S*1	Rock of any characteristic, either shalelike or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second or by other appropriate means of classification.	1.0
	or	
	Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels or stiff clays.	
*S*2	Deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels or stiff clays.	1.2
*S*3	Soft- to medium-stiff clays and sands characterized by 30 feet or more of soft- to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.	1.5
*S*4	More than 70 feet of soft clays or silts characterized by a shear wave velocity less than 400 feet per second.	2.0

1. **Steel elements.** Steel elements shall be classified by structural member type (e.g., rolled or build-up, material grade, and general properties). The survey shall note the presence of degradation or indications of plastic deformation, integrity of surface coatings, and signs of any past movement. For degraded elements, the lost material thickness and reduction of cross-sectional area and moment of inertia shall be determined. Visual inspection of welds shall be per American Welding Society D1.1, "Structural Welding Code-Steel." Structural bolts shall be verified to be in proper configuration and tightened as required in the AISC Steel Construction Manual. Rivets shall also be verified to be in proper configuration and in full contact, with "hammer sounding" conducted on random rivets to ensure they are functional. Nondestructive testing methods, such as dye penetrant and magnetic particle testing, acoustic emission, radiography and ultrasound shall be used when visual inspection identifies degradation or when a particular element or connection is critical to seismic resistance and requires further verification. For buildings in which archaic cast and wrought irons are employed, additional investigations to confirm ductility and impact resistance shall be conducted.
2. **Concrete elements.** The configuration and dimensions of primary and secondary structural elements shall be established. The configuration and condition of reinforcing steel shall be assessed, through removal of concrete cover and direct visual inspection, and through nondestructive inspection using electromagnetic, radiographic and other methods. Critical parameters of

(TEXT CONTINUES ON PAGE 67)

the reinforcing system, such as lap splice length, presence of hooks, development within concrete, degree of corrosion and integrity of the construction shall be established in sufficient detail to perform the structural evaluation.

3. **Masonry elements.** The configuration and dimensions of masonry elements shall be established. The configuration and condition of reinforcing-steel shall be assessed, through removal of masonry cover and direct visual inspection, and through nondestructive inspection using electromagnetic, radiographic and other methods. Critical parameters of the reinforcing system, such as lap splice length, presence of hooks, development within concrete, degree of corrosion and integrity of the construction shall be established in sufficient detail to perform the structural evaluation.

4. **Wood elements.** The configuration and dimensions of wood elements; the connections between wood elements; and the connections between wood and other structural components or elements such as concrete or masonry walls shall be established. The configuration and condition of wood members, including size, type, grade, condition and quality shall be assessed, through removal of finish materials, and examination of unfinished areas such as attics, crawl spaces and basements. Critical connections and elements shall be visually inspected, using invasive procedures or removal of finishes where necessary. For shear walls, select locations shall be exposed to allow evaluation of sheathing material, nail size, spacing and installation (e.g., overdriven or nails that miss or split the framing members). The base connections of shear resisting elements shall be inspected and evaluated for their adequacy to connect the base of the structure to the foundation or structure below.

5. **Foundation elements.** In the absence of dependable construction drawings, determination of the size and detailing of the foundation system requires invasive procedures. The evaluator shall select representative footings for exposure to establish footing size and depth. Conservative assumptions regarding the reinforcement may be made considering code requirements and local practice at the time of the design. In the absence of evidence to the contrary, it may be assumed that the foundation elements were adequately designed to resist actual gravity loads to which the building has been subjected.

2.1.2.2 Material properties. The building evaluation shall be based on the strength and deformation properties of the existing materials and components. The strength of existing components shall be calculated using data on their configuration, obtained from the original construction documents, supplemented by field observations and the test values of material properties. Where such effects may have a deleterious effect on component or structural behavior, allowances shall be made for the likely effects of strain hardening or degradation. Test values may be obtained from samples extracted from the structure, or from original materials and compliance certificates. The Office will determine the adequacy of the testing program.

2.1.2.2.1 Nonconforming buildings with construction documents. The material properties for nonconforming buildings for which original construction documents of sufficient detail are available shall be confirmed by testing or from acceptable original materials and compliance certificates. If original materials and compliance certificates are available, they must provide the information specified in Items 1 through 4 of this section to be considered acceptable.

1. **Steel elements.** The following properties are required for each member type (e.g., beams, columns, braces) and each steel grade used in the structure:
   a) Ultimate tensile and yield capacities;
   b) Modulus of elasticity; and
   c) Deformation characteristics including mode of failure.
2. **Concrete elements.** The following material properties are required for each member type (e.g., beams, columns, walls) in the structure:
   a) Concrete compressive strength;
   b) Concrete unit weight;
   c) Concrete modulus of elasticity;
   d) Reinforcing steel tensile yield point;
   e) Reinforcing steel modulus of elasticity;
   f) Reinforcing steel chemical composition and carbon equivalent; and
   g) Reinforcing steel surface deformations.
3. **Masonry elements.** The following material properties are required for each type of masonry in the structure:
   a) Masonry compressive strength;
   b) Masonry unit weight;
   c) Masonry modulus of elasticity;
   d) Reinforcing steel tensile yield point;
   e) Reinforcing steel modulus of elasticity;
   f) Reinforcing steel chemical composition and carbon equivalent; and
   g) Reinforcing steel surface deformations.
4. **Wood elements.** The following material properties are required for each type of wood element in the structure:
   a) Identification of Wood Species, and
   b) Grade Material. (Note: This may be established by visual inspection or stamped labels on the element.)

2.1.2.2.2 Nonconforming buildings without construction documents. The material properties for nonconforming buildings for which original construction documents of sufficient detail are unavailable shall be confirmed by testing. The number and location of tests shall be selected so as to provide sufficient information to adequately define the existing condition of materials in the building. The evaluator shall determine the number and location of tests. The test locations shall be located throughout the entire building in those components which provide the primary path of lateral force resistance.

2.2 Selection and use of evaluation statements.

2.2.1 Identification of building type. The evaluator shall determine the building type using the following procedure:

1. Identify the lateral-force-resisting system using text and drawings, including whatever components are available and effective to constitute a system. Prepare floor and roof plans, and elevations and sketches of the lateral-force-resisting system.
2. Select one or more of the 15 common building types which best characterize the structure (see Sections 2.2.2 and 2.2.3 below). Structures with multiple lateral force resisting systems (different lateral systems in orthogonal directions, or structures where the system changes from level to level) may require the use of two or more building types. In the case of hybrid structures or other buildings that cannot be adequately classified using the 15 building types, the alternative analysis procedure shall be used, or the building shall be placed in SPC "1."
3. Reproduce from the Appendix the list of evaluation statements. These statements shall be used for all types of buildings. Some statements on the list may not be appropriate. These statements may be marked "NA" as "not applicable." The Appendix also contains the set of evaluation statements that address foundations and geologic site hazards, and nonstructural elements.

2.2.2 Using the general procedure. The general procedure involving use of the set of evaluation statements presented in the Appendix consists of the following steps:

1. Evaluate the basic building system according to the evaluation statements in Article 3;
2. Evaluate the vertical systems resisting lateral forces according to Article 4 (moment frames), Article 5 (shear walls) or Article 6 (braced frames) as appropriate. For buildings with a combination of vertical systems, each system in the building must be evaluated;
3. Evaluate the diaphragm or horizontal bracing system according to Article 7;
4. Evaluate the structural connections according to Article 8;
5. Evaluate the foundation and possible geologic site hazards according to Article 9;
6. Evaluate the nonstructural elements that involve immediate life-safety issues according to Article 10; and
7. Evaluate the critical nonstructural components and systems according to Article 11.

If a statement is found to be true, the condition being evaluated is acceptable according to the criteria of these regulations, and the issue may be set aside. If a statement is found to be false, a condition exists that needs to be addressed further, using the specified analysis procedures. Analysis procedures are given in Section 2.4. Each statement includes a reference to a particular section in Articles 3 through 10 where additional procedures for the resolution of the issues are given. The evaluator shall assemble the list of deficiencies and the results of the analysis and proceed to the final evaluation in Section 2.5.

2.2.3 Common building types. The evaluator shall determine the type(s) of building being evaluated, choosing from among the following 15 common types:

1. **Building Type 1—Wood, light frame.** These buildings are typically small structures of one or more stories. The essential structural character of this type is repetitive framing by wood joists on wood studs. Loads are light and spans are small. These buildings may have relatively heavy chimneys and may be partially or fully covered with veneer. Lateral loads are transferred by diaphragms to shear walls. The diaphragms are roof panels and floors. Shear walls are exterior walls sheathed with plank siding, stucco, plywood, gypsum board, particle board or fiberboard. Interior partitions are sheathed with plaster or gypsum board.
2. **Building Type 2—Wood, commercial and industrial.** These are buildings with a floor area of 5,000 square feet or more and with few, if any, interior bearing walls. The essential structural character is framing by beams on columns. The beams may be glulam beams, steel beams or trusses. Lateral forces usually are resisted by wood diaphragms and exterior walls sheathed with plywood, stucco, plaster or other paneling. The walls may have rod bracing. Large exterior wall openings often require post-and-beam framing. Lateral force resistance on those lines may be achieved with steel rigid frames or diagonal bracing.
3. **Building Type 3—Steel moment frame.** These buildings have a frame of steel columns and beams. Lateral forces are resisted by the development of flexural forces in the beams and columns. In some cases, the beam-column connections have very small moment resisting capacity but, in other cases, the connections of some of the beams and columns were designed to fully develop the member capacities. Lateral loads are transferred by diaphragms to moment resisting frames. The diaphragms can be of almost any material. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction, and the frames consist of lines of strong columns and their intervening beams.
4. **Building Type 4—Steel braced frame.** These buildings are similar to Type 3 buildings except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.
5. **Building Type 5—Steel light frame.** These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of

tension-only rod bracing, or a combination of panels and bracing.

6. **Building Type 6—Steel Frame with concrete shear walls.** The shear walls in these buildings are cast-in-place concrete and may be bearing walls. The steel frame is designed for vertical loads only. Lateral loads are transferred by diaphragms of almost any material to the shear walls. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections. In "dual" systems, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidities. In this case, the walls would be evaluated under this building type and the frames would be evaluated under Type 3, Steel Moment Frames.
7. **Building Type 7—Steel frame with infill shear walls.** This is one of the older type of buildings. The infill walls usually are offset from the exterior frame members, wrap around them, and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels act as a diagonal compression strut between the intersections of the moment frame. If the walls do not fully engage the frame members (i.e., lie in the same plane), the diagonal compression struts will not develop. The peak strength of the diagonal strut is determined by the tensile stress capacity of the masonry panel. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The analysis shall be based on published research and shall treat the system as a composite of a frame and the infill. An analysis that attempts to treat the system as a frame and shear wall is not permitted.
8. **Building Type 8—Concrete moment frame.** These buildings are similar to Type 3 buildings except that the frames are of concrete. There is a large variety of frame systems. Older buildings may have frame beams that have broad shallow cross sections or are simply the column strips of flat-slabs.
9. **Building Type 9—Concrete shear walls.** The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low but reinforcing is light. Remodeling that entailed adding or enlarging the openings for windows and doors may critically alter the strength of the modified walls. In newer buildings, the shear walls often are limited in extent, generating the need for boundary members and additional design consideration of overturning forces.
10. **Building Type 10—Concrete frame with infill shear walls.** These buildings are similar to Type 7 buildings except that the frame is of reinforced concrete. The analysis of this building is similar to that recommended for Type 7 except that the shear strength of the concrete columns, after cracking of the infill, may limit the semi-ductile behavior of the system. Research that is specific to confinement of the infill by reinforced concrete frames shall be used for the analysis.
11. **Building Type 11—Precast/tilt-up concrete walls with lightweight flexible diaphragm.** These buildings have a wood or metal deck roof diaphragm that distributes lateral forces to precast concrete shear walls. The walls are thin but relatively heavy while the roofs are relatively light. Tilt-up buildings often have more than one story. Walls can have numerous openings for doors and windows of such size that the wall behaves more like a frame than a shear wall.
12. **Building Type 12—Precast concrete frames with concrete shear walls.** These buildings contain 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 usually are cast-in-place concrete. Welded steel inserts often are used to interconnect precast elements. Lateral loads are resisted by precast or cast-in-place concrete shear walls.
13. **Building Type 13—Reinforced masonry bearing walls with wood or metal deck diaphragms.** These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck with or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns; steel beams are supported by steel columns.
14. **Building Type 14—Reinforced masonry bearing walls with precast concrete diaphragms.** These buildings have bearing walls similar to those of Type 13 buildings, but the roof and floors are composed of precast concrete elements such as planks or tee-beams, and the precast roof and floor elements are supported on interior beams and columns of steel or concrete (cast-in-place or precast). The precast horizontal elements may have a cast-in-place topping.
15. **Building Type 15—Unreinforced masonry (URM) bearing wall buildings.** These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In buildings built after 1950, unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. The perimeter walls, and possibly some interior walls, are unreinforced masonry. 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. Unreinforced masonry bearing wall buildings

(TYPE 15) shall be assigned to SPC 1. No further analysis is required.

2.3 Follow-up field work. The first assessment of the evaluation statements may indicate a need for more information about the building. The evaluator shall make additional site visits, performing the necessary surveys and tests to complete the evaluation.

2.4 Analysis of the building. The general requirements for building analysis (including the determination of force level, horizontal distribution of lateral forces, accidental torsion, interstory drift and overturning) are summarized in this section. For cases where dynamic analysis is required, the general requirements are given in Section 2.4.10.

2.4.1 Scope of analysis. When an evaluation statement is false and requires further analysis, the evaluator shall provide appropriate analyses that will cover the statement requirements. For the analysis, the evaluator will:

1. Calculate the building weights;
2. Calculate the building period;
3. Calculate the lateral force on the building;
4. Distribute the lateral force over the height of the building;
5. Calculate the story shears and overturning moments;
6. Distribute the story shears to the vertical resisting elements in proportion to their relative stiffness;
7. Examine the individual elements as required by the evaluation statements:
   a. Load and reaction diagrams for diaphragms and for the vertical resisting elements;
   b. Shearing stresses and chord forces in the diaphragm;
   c. Vertical components (walls and frames) and find the story deflections, member forces and deflections; and
   d. Total forces or deflections according to the specified load combinations.

For moment frames consisting of beams and columns, the distribution of story shears to the vertical lateral-force-resisting elements in that story may be in proportion to their relative stiffness. In multistory frame-shear wall structures or in structures where the vertical resisting elements have significantly different lateral stiffnesses, or where the stiffnesses of the vertical resisting elements change significantly over the height of the structure, an analysis of the entire structure under the prescribed lateral loads shall be performed.

2.4.2 Demand. All building components evaluated shall resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads. The following load combinations shall be used:

$$Q = 1.1\,Q_D + Q_L + Q_S \pm Q_E \qquad (2\text{-}1)$$

or

$$Q = 0.9Q_D \pm Q_E \qquad (2\text{-}2)$$

where:

Q = the effect of the combined loads.

Q_D = the effect of dead load.

Q_E = the effect of seismic forces.

Q_L = the effective live load is equal to 25 percent of the unreduced design live load but not less than the actual live load.

Q_S = the effective snow load is equal to either 70 percent of the full design snow load or, where conditions warrant and are approved by OSHPD, not less than 20 percent of the full design snow load except that, where the design snow load is less than 30 pounds per square foot, no part of the load need be included in seismic loading.

The seismic portion of the demand *(Q_E)* is obtained from analysis of the building using the seismic base shear *(V)* from Equation 2-3.

2.4.3 Seismic analysis of the building.

2.4.3.1 Base shear. The seismic base shear determined from Equation 2-3 is the basic seismic demand on the building. Element forces and deflections obtained from analysis based on this demand are the element demands *(Q_E)* to be used in the load combinations of Equations 2-1 and 2-2. The demands are modified in some cases as discussed in Section 2.4.11.

The seismic base shear *(V)* in a given direction shall be determined as follows:

$$V = C_s\, W \qquad (2\text{-}3)$$

where:

C_s = the seismic design coefficient determined by Equation 2-4 or 2-5.

W = the total dead load and applicable portions of the following:

- In storage and warehouse occupancies, a minimum of 25 percent of the floor live;
- Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater;
- Total operating weight of all permanent equipment; and
- The effective snow load as defined in Section 2.4.2.

The seismic coefficient (C_s) for existing buildings shall be determined as follows:

$$C_s = 0.67\left(\frac{1.2A_vS}{RT^{2/3}}\right) = \frac{0.80A_vS}{RT^{2/3}} \qquad (2\text{-}4)$$

where:

A_v = the peak velocity-related acceleration coefficient given in Figures 2.1 and 2.1a.

R = a response modification coefficient from Table 2.4.3.1.

S = the site coefficient given in Table 2.1. In locations where the soil properties are not known in sufficient detail to determine the Soil Profile Type S_3 shall be used. Soil Profile Type S_4 need not be assumed unless OSHPD determines that Soil Profile Type S_4 may be present at the site, or in the event the Soil Profile Type S_4 is established by the geotechnical engineer.

TABLE 2.4.3.1—RESPONSE COEFFICIENTS[1]

R	C_d	SYSTEM
		Bearing wall systems
6.5	4	Light-framed walls with shear panels
4.5	4	Reinforced concrete shear walls
3.5	3	Reinforced masonry shear walls
4	3.5	Concentrically braced frames
1.25	1.25	Unreinforced masonry shear walls
		Building frame systems
8	4	Eccentrically braced frames, moment-resisting connections at columns away from link
7	4	Eccentrically braced frames, nonmoment-resisting connections at columns away from link
7	4.5	Light-framed walls with shear panels
5	4.5	Concentrically braced frames
5.5	5	Reinforced concrete shear walls
4.5	4	Reinforced masonry shear walls
3.5	3	Tension-only braced frames
1.5	1.5	Unreinforced masonry shear walls
		Moment-resisting frame system
8	5.5	Special moment frames of steel
8	5.5	Special moment frames of reinforced concrete
4	3.5	Intermediate moment frames of reinforced concrete
4.5	4	Ordinary moment frames of steel
2	2	Ordinary moment frames of reinforced concrete
		Dual system with a special moment frame capable of resisting at least 25 percent of prescribed seismic forces.
		Complementary seismic-resisting elements
8	4	Eccentrically braced frames, moment-resisting connections at columns away from link
7	4	Eccentrically braced frames, nonmoment-resisting connections at columns away from link
6	5	Concentrically braced frames
8	6.5	Reinforced concrete shear walls
6.5	5.5	Reinforced masonry shear walls
8	5	Wood sheathed shear panels
		Dual system with an intermediate moment frame of reinforced concrete or an ordinary moment frame of steel capable of resisting at least 25 percent of prescribed seismic forces.
		Complementary seismic-resisting elements
5	4.5	Concentrically braced frames
6	5	Reinforced concrete shear walls
5	4.5	Reinforced masonry shear walls
7	4.5	Wood sheathed shear panels
		Inverted pendulum structures
2.5	2.5	Special moment frames of structural steel
2.5	2.5	Special moment frames of reinforced concrete
1.25	1.25	Ordinary moment frames of structural steel

[1]Some building systems such as precast moment resisting frames are not listed in Table 2.4.3.1. When an unlisted building system must be evaluated, the evaluator shall perform an alternate analysis per Section 2.7 or place the building in SPC 1.

T = the fundamental period of the building.

The value of C_s need not be greater than:

$$C_s = 0.85\left(\frac{2.5\,A_a}{R}\right) = \frac{2.12A_a}{R} \qquad (2\text{-}5)$$

where:

A_a = the effective peak acceleration coefficient given in Figures 2.1 and 2.1a.

2.4.3.2 Period. For use in Equation 2-4, the value of T shall be calculated using one of the following methods:

Method 1. The value of T may be taken to be equal to the approximate fundamental period of the building (T_a) determined as follows:

a. For buildings in which the lateral-force-resisting system consists of moment-resisting frames capable of resisting 100 percent of the required lateral force and such frames are not enclosed or adjoined by more rigid components tending to prevent the frames from deflecting when subjected to seismic forces:

$$T_a = C_T h_n^{3/4} \qquad (2\text{-}6a)$$

where:

C_T = 0.035 for steel frames.

C_T = 0.030 for concrete frames.

h_n = the height in feet above the base to the highest level of the building.

b. As an alternate for concrete and steel moment-resisting-frame buildings of 12 stories or fewer with a minimum story height of 10 feet, the equation $T_a = 0.10N$, where N= the number of stories, may be used in lieu of Equation 2-6a.

c. For all other buildings,

$$T_a = \frac{0.05\,h_n}{\sqrt{L}} \qquad (2\text{-}6b)$$

where:

L = the overall length (in feet) of the building at the base in the direction under consideration.

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

$$T = 2\pi\sqrt{\frac{\Sigma(w_i d_i^2)}{g\Sigma(f_i d_i)}} \qquad (2\text{-}7)$$

TABLE 2.4.3.2—COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

A_v	C_a
0.4	1.2
0.3	1.3
0.2	1.4

The values of f_i represent any lateral force, associated with weights w_i, distributed approximately in accordance with the principles of Equations 2-8, 2-9 and 2-10 or any other rational distribution. The elastic deflections, d_i, should be calculated using the applied lateral forces, f_i. The period used for computation of C_s shall not exceed C_aT_a, where C_a is given in Table 2.4.3.2.

2.4.3.3 Direction of seismic forces. Assume that seismic forces will come from any horizontal direction. The forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure except as discussed in Section 2.4.3.5.

2.4.3.4 Uplift. The beneficial effects of uplift at the foundation soil level may be considered, using the alternative analysis procedure.

2.4.3.5 Orthogonal effects. The critical load effect due to direction of application of seismic forces on the building may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength should be used.

> **Exception:** Diaphragms and components of the seismic resisting system utilized in only one of the two orthogonal directions need not be designed for the combined effects.

2.4.3.6 Combinations of structural systems. When combinations of structural systems are incorporated into the same structure, the following requirements shall be satisfied:

1. **Vertical combinations.**

    1.1 Structures not having the same structural system throughout their height shall be evaluated using the dynamic lateral force procedure.

    Exceptions:

    1. Structures five stories or less without stiffness and strength irregularities may be evaluated using the equivalent lateral force procedures; and
    2. Structures conforming to Section 2.4.3.6.2, below.

    1.2 A two-stage analysis may be used if a structure contains a relatively rigid base supporting a flexible upper portion and both portions considered separately can be classified as regular structures. The rigid base shall have a calculated natural period in each direction of not more than 0.06 seconds. The periods shall be evaluated using Eq. 2-7, or its equivalent, considering the total mass of the flexible upper portion concentrated at the top of the rigid base. The flexible upper portion shall be evaluated as a separate structure supported laterally by the rigid base. The rigid base shall be evaluated as a separate structure. The reactions of the flexible upper portion shall be applied at the top of the rigid base, amplified by the ratio of the R and C_d factors of the superstructure divided by those for the base structure. The values of R and C_d for the base structure shall be greater than or equal to those used for the superstructure. The total lateral force on the base shall include the forces determined for the base itself.

2. **Combinations along different Axes.** If a building has a bearing wall system in only one direction, the value of R used for systems in the other direction shall not be greater than that used for the bearing wall system.

2.4.3.7 Vertical distribution of forces. The lateral force *(F_x)*, induced at any level, shall be determined as follows:

$$F_x = C_{vx}V \tag{2-8}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{2-9}$$

where:

C_{vx} = vertical distribution factor.

h_i and h_x = the height (feet) from the base to Level I or x.

k = an exponent related to the building period as follows:
For buildings having a period of 0.5 second or less, k = 1.
For buildings having a period of 2.5 seconds or more, k = 2.
For buildings having a period between 0.5 and 2.5 seconds, k may be taken as 2 or may be determined by linear interpolation between 1 and 2.

V = total design lateral force or shear at the base of the building.

w_i and w_x = the portion of the total gravity load of the building *(W)* located or assigned to Level I or x.

2.4.3.8 Horizontal distribution of shear. The story shear, *(V_x)*, shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm.

2.4.3.9 Horizontal torsional moments. The increased shears resulting from horizontal torsion where diaphragms have the capability to transmit that torsion shall be evaluated. The accidental torsional moment shall 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 shall be five percent of the dimension at that level measured perpendicular to the direction of the applied force. For each element, the most severe loading shall be considered.

2.4.3.10 Overturning. Every structure shall be capable of resisting the overturning effects caused by earthquake forces specified. At any level, the overturning moments to be resisted shall be estimated using those seismic forces *(F_t and F_x)* that act on levels above the level under consideration. At any level, the incremental changes of the overturning moment shall be distributed to the various resisting elements in the same proportion as distribution of the horizontal shears to those elements. The foundations of buildings (but not the connection of the building to the foundation), except inverted pendulum structures, shall be evaluated for the foundation overturning design moment *(M_f)* at the foundation-soil interface determined using the overturning moment at the base with an overturning moment reduction factor of 0.75.

2.4.3.11 *P*-delta effects. The resulting member forces and moments and the story drifts induced by *P*-delta effects shall be considered in the evaluation of overall structural frame stability. *P*-delta need not be considered if the drift satisfies the "Quick Check for Drift" given in Section 2.4.7.

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

2.4.3.12.1 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 shall 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 capacities multiplied by a factor of 2.0 may be used, except that values for sliding friction may not be increased.

2.4.3.12.2 Structural materials. The strength of concrete foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall be determined from the provisions of ACI 318. Reductions to foundation component capacities shall be made where components do not meet the requirements of ACI 318.

2.4.4 Deformation and drift. When deformations and drift limits need to be checked, such as for frames failing the "Quick Check of Drift" and slender seismic resisting systems of any type, compute the elastic deformations caused by the required forces and then multiply by the factor C_d to determine the total deformations. Interstory drifts shall not exceed $0.0133h_{sx}$, where h_{sx} is the story height below level x . For purposes of this drift analysis only, it is permissible to use the computed fundamental period (T) of the building without the upper bound limitation specified in Section 2.4.3.2 when determining drift level seismic design forces.

2.4.5 Demand on diaphragms. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements as determined by the evaluator. Permissible deflection permits the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

Floor and roof diaphragms shall be designed to resist a minimum force equal to $0.5A_V$ times the weight of the diaphragm and other elements attached to the building plus the portion of the seismic shear force at that level, (V_x), required to be transferred to the components of the vertical seismic-resisting system because of offsets or changes in stiffness of the vertical components above and below the diaphragm.

Diaphragms shall provide for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm as prescribed in Section 3.6.4 of the 1994 *NEHRP Recommended Provisions.*

2.4.6 Demand on parts and portions of the building. 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, shall be evaluated to verify that they are capable of resisting the seismic forces specified below. All attachments or appendages, including anchorages and required bracing, shall be evaluated for seismic forces. Nonrigid equipment, the structural failure of which would cause a life-safety hazard, also shall be evaluated.

Each element or component evaluated shall be capable of resisting a total lateral seismic force, F_p, where:

$$F_p = 0.67(A_V C_C W_C) \qquad (2\text{-}10)$$

where:

A_v = the velocity-related acceleration coefficient given in Figures 2.1 and 2.1a.

C_c = a coefficient given in Table 2.4.6.

W_c = the weight of the element or component.

The NPC of the building shall be determined using the procedures in Article 11.

TABLE 2.4.6—SEISMIC COEFFICIENT, C_c

		C_c
Parts of structure	Walls:	
	Unbraced (cantilevered parapets and walls)	2.4
	Other exterior walls at and above the ground floor	0.9
	All interior bearing and nonbearing walls and partitions	0.9
	Masonry or concrete fences over 6 feet high	0.9
	Penthouse (except where framed by an extension of the building frame)	0.9
	Connections for prefabricated structural elements other than walls with force applied at the center of gravity	0.9
Nonstructural components	Exterior and interior ornamentations and appendages	2.4
	Chimneys, stacks, trussed towers and tanks:	
	Supported on or projecting as an unbraced cantilever above the roof more than one-half its total height	2.4
	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	0.9
	Mechanical, plumbing and electrical equipment	0.9
	Anchorage for suspended ceilings and light fixtures	0.9

2.4.7 Quick checks of strength and stiffness. Evaluation statements may require quick check estimates of the strength and stiffness of the building.

To check the average shear stress or drift for upper stories in addition to the first story, the story shear for an upper story may be approximated as follows:

$$V_j = \left(\frac{n+j}{n+1}\right)\left(\frac{W_j}{W}\right)1.2V \qquad (2\text{-}11)$$

where:

- j = number of story level under consideration.
- n = total number of stories above ground level.
- V = base shear from Equation 2-3.
- V_j = maximum story shear at story Level j.
- W = total seismic dead load.
- W_j = total seismic dead load of all stories above Level j (see Section 2.4.1).

2.4.7.1 Story drift for moment Frames.The following equation for the drift ratio is applicable only to regular, multistory, multibay frames with columns continuous top and bottom:

$$DR = \left(\frac{k_b + k_c}{K_b \bullet K_c}\right)\left(\frac{h}{12E}\right)V_c C_d \qquad (2\text{-}12)$$

where:

- C_d = deflection amplification factor from Table 2.4.3.1.
- DR = drift ratio = interstory displacement divided by interstory height.
- E = modulus of elasticity (ksi).
- h = story height (in.).
- I = moment of inertia (in.^{4}).
- k_b = I/L for the beam.
- k_c = I/h for the column.
- L = center-to-center length (in.).
- V_c = shear in the column (kips).

For reinforced concrete frames, use appropriate cracked section properties pursuant to ACI 318-95 or later. For other configurations of frames, compute the drift ratio from the principles of structural mechanics.

2.4.7.2 Shearing stress in concrete frame columns. The equation for a quick estimate of the average shearing stress, (v_{avg}), in the columns of concrete frames is as follows:

$$v_{avg} = \left(\frac{n_c}{N_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \qquad (2\text{-}13)$$

where:

- A_c = summation of the cross-sectional area of all columns in the story under consideration.
- n_c = total number of columns.
- n_f = total number of frames in the direction of loading.
- V_j = story shear from Equation 2-11.

Equation 2-13 assumes that nearly all of the columns in the frame have similar stiffness. For other configurations of frames, compute the shear stress in the concrete columns from the principles of structural mechanics.

2.4.7.3 Shearing stress in shear walls. The equation for a quick estimate of the average wall shear stress *(v_{avg})* is as follows:

$$v_{avg} = \frac{V_j}{A_w} \qquad (2\text{-}14)$$

where:

- A_w = summation of the horizontal cross-sectional area of all shear walls in the direction of loading. The wall area shall be reduced by the area of any openings. For masonry walls, use the net area. For wood-framed walls, use the length rather than the area.
- V_j = story shear at the level under consideration determined from Equation 2-11.

The allowable stresses for the various types of shear wall building are given in Section 5.1 for concrete shear walls, Section 5.3 for reinforced masonry shear walls, Section 5.4 for unreinforced masonry shear walls and Section 5.6 for wood shear walls.

2.4.7.4 Diagonal bracing. The equation for a quick estimate of the average axial stress in the diagonal bracing *(f_{br})* is as follows:

$$f_{br} = \left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \qquad (2\text{-}15)$$

where:

- A_{br} = the average area of a diagonal brace (in.^{2}).
- L_{br} = average length of the braces (ft).
- N_{br} = number of braces in tension and compression if the braces are designed for compression; if not, use the number of braces in tension, if the braces are not designed for compression.
- s = average span length of braced spans (ft).
- V_j = maximum story shear at each level (kips).

2.4.8 Procedure for evaluating unreinforced masonry bearing wall buildings. Unreinforced masonry bearing wall buildings shall automatically be placed in SPC 1.

2.4.9 Element capacities. Calculate element capacities on the ultimate-strength basis of the 1994 *NEHRP Recommended Provisions.*

When calculating capacities of deteriorated or damaged elements, the evaluator shall make appropriate reductions in the material strength, the section properties and any other aspects of the capacity affected by the deterioration.

2.4.9.1 Wood. The basic document is Chapter 9 of the 1994 *NEHRP Recommended Provisions,* as modified in Section 5.6 of these regulations.

2.4.9.2 Steel. The basic document is Chapter 5 of the 1994 *NEHRP Recommended Provisions,* as modified in Articles 4 and 6 of these regulations.

2.4.9.3 Concrete. The basic document is ACI 318-89. Because this document is on an ultimate-strength basis, the 1994 *NEHRP Recommended Provisions* specifies special load factors that include the factor of 1.0 for earthquake effects (see Equations 2-1 and 2-2).

2.4.9.4 Masonry. The basic document is Chapter 8 of the 1994 *NEHRP Recommended Provisions,* as modified in Article 5 of these regulations.

2.4.10 Dynamic analysis. Unless otherwise noted, the procedures given in Articles 3 through 10 use the equivalent lateral force procedure. The use of a dynamic analysis procedure is required for the following:

1) Buildings 240 feet or more in height;
2) Buildings with vertical irregularities caused by significant mass or geometric irregularities;
3) Buildings where the distribution of the lateral forces departs from that assumed in the equivalent lateral force procedure; and
4) Where required by the evaluation statements in Articles 3 through 10.

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

2.4.10.1 Ground motion. The ground motion representation shall be an elastic response spectra developed for mean values for the specific site, in accordance with the procedures in Title 24, Section 1629A.2 of 1995 *California Building Code* (CBC) or equivalent provision in later version of the CBC.

2.4.10.2 Mathematical model. A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to calculate the significant features of its dynamic response. A three-dimensional model shall be used when the dynamic analysis involves a structure with an irregular plan configuration and rigid or semirigid diaphragms.

2.4.10.3 Analysis procedure.

2.4.10.3.1 Response spectrum analysis. An elastic dynamic analysis of a structure shall use the peak dynamic response of all modes having a significant contribution to total structural response. This requirement may be satisfied by demonstrating that for the modes considered, at least 90% of the participating mass of the structure is included in the calculation of response in each principal horizontal direction. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve that corresponds to the modal periods. Maximum modal contributions shall be combined in a statistical manner using recognized combination methods to obtain an approximate total structural response.

2.4.10.3.2 Scaling of results. When the base shear for a given direction is less than that required by the equivalent lateral force procedure, the base shear shall be increased to the value prescribed in that procedure. All corresponding response parameters, including deflections, member forces and moments, shall be increased proportionately.

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 procedure. All corresponding response parameters, including deflections, member forces, and moments, may be decreased proportionately.

2.4.10.3.3 Post-yield analyses. Post-yield analyses of a simplified model of the building may be made to estimate the nonlinear displacements of the structural system. If the analyses is made with a two-dimensional planar model, the additive torsional displacement shall be established through methods that are equivalent to those used for response spectra analyses.

The displacements or rotations of structural members estimated by the post-yield analysis shall be compared with relevant experimental data to determine the adequacy of the member or system.

2.4.10.4 Torsion. The analysis shall account for torsional effects, including accidental torsional effects, as prescribed in Section 2.4.3.9. Where three-dimensional models are used for analysis, effects of accidental torsion shall 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 2.4.3.9.

2.4.11 Acceptance criteria. The elements to be analyzed are specified in the procedures given in Articles 3 through 10. The total demand, Q, is calculated by Equation 2-1 or 2-2 as modified below. The capacity, C, is calculated according to the procedures of Section 2.4.9. The basic acceptance criterion is:

$$Q \le C \qquad (2\text{-}17)$$

Where elements or portions of a lateral force resisting system are expected to behave in a less ductile manner than the system as a whole, the term Q_E in Equation 2-1 or 2-2 shall be modified or special calculations be made to account for the different failure modes of the various elements. Modification of Q_E, and special calculation procedures and when they shall be used, are described in Articles 3 through 8.

If all significant elements meet the basic acceptance criteria as specified herein, no further analysis is needed.

2.4.12 Assessment of element deficiencies. The result of the checks specified in Articles 3 through 10 will show whether or not the elements meet the requirements of the 1994 *NEHRP Recommended Provisions* as modified herein.

For those elements not meeting the specified acceptance criteria, the relative hazard or seriousness of the deficiencies shall be assessed. Deficiencies shall be ranked according to:

1) Degrees of "overstress" (both total and seismic);
2) Element importance in the load path; and
3) Building, ductile and element stability.

2.5 Final evaluation.

2.5.1 Review the statements and responses. Upon completion of the analysis and field work, the evaluator shall review the evaluation statements and the responses to the statements to ensure that all of the concerns have been addressed.

2.5.2 Assemble and review the results of the procedures. Upon completion of the procedures given in Articles 3 through 10, the evaluator shall assemble and review the results.

2.5.2.1 *Q* versus *C*. The criterion $Q \le C$ is an indication of whether an element meets the requirements of the 1994 *NEHRP Recommended Provisions* as modified for these regulations. However, because Q involves gravity effects, the ratio of Q to C for an element must be considered in light of the seismic demand versus capacity in order to fully determine the seriousness of the earthquake hazard.

2.5.2.2 D_E/C_E Ratios. The severity of the deficiencies shall be assessed by listing the D_E/C_E ratios in descending order. The element with the largest value is the weakest link in the building. If the element can fail without jeopardizing the building, then the SPC may be based upon the element with the next lower ratio, and so on. Failure of an element will not jeopardize the building provided an alternate load path (neglecting the failed element) exists, and the vertical and lateral stability of the structure, or portions of the structure, is not impaired. The presence of an element with a D_E/C_E greater than one, where failure of that element will jeopardize the stability of the building or element, requires that nonconforming buildings be placed in SPC 1. For conforming buildings, see the appropriate evaluation statement.

2.5.2.3 Qualitative issues. Some of the procedures identify specific deficiencies without any calculation. These deficiencies will automatically place buildings in SPC 1, 3 or 4.

2.5.3 Final evaluation. The final evaluation will place the building in the appropriate the SPC (Table 2.5.3), based on a review of the qualitative and quantitative results of the procedures and the list of deficiencies. In general, an unmitigated "false" answer to an evaluation statement will lower the SPC of the Building. A "false" evaluation statement may be considered mitigated if the building, element or component is justified using the procedure outlined in the evaluation statement, or the effects of the condition are incorporated in the overall evaluation, as described in Section 2.5.2.2. Alternatively, the SPC rating of a building may be assigned by the Office on the basis of a collapse probability assessment performed in accordance with Section 1.4.5.1.2.

2.5.3.1 Conforming buildings. Conforming buildings, other than those of welded steel moment frame construction (Building Type 3 and possibly Building Types 4 and 6, if a dual system is present), without any unmitigated "false" evaluation statements shall be placed in SPC 5. Other conforming buildings shall be placed in the lowest SPC directed by the evaluation statements.

2.5.3.2 Nonconforming buildings. An unmitigated "False" answer to any evaluation statement shall result in nonconforming buildings being placed in SPC 1, unless directed otherwise by the procedures for that particular evaluation statement. All other nonconforming buildings shall be placed in SPC 2.

2.6 The final report. The report shall include the following elements:

1. A description of the building, including photographs, and sketches of the lateral-force-resisting system using an OSHPD approved format;
2. The set of statements from the Appendix, with a synopsis of the investigation and supporting calculations that were made;
3. A list of the deficiencies that must be remedied in order to change statement responses from false to true;
4. The SPC for the building, with comments on the relative importance of the deficiencies; and
5. The NPC for the building.

2.7 Alternative analysis. The owner of a building may elect to perform an Alternative Analysis, to evaluate a structure in more detail than that provided by the evaluation procedures specified in these regulations. The methodology of an Alternative Analysis must be approved in advance by OSHPD, and shall meet the following criteria:

TABLE 2.5.3—STRUCTURAL PERFORMANCE CATEGORIES (SPC)

SPC	DESCRIPTION
SPC 1	Buildings posing a significant risk of collapse and a danger to the public. These buildings must be brought up to the SPC 2 level by January 1, 2008, or be removed from acute care service. Where the office has performed a collapse probability assessment, buildings with Probability of Collapse greater than 0.75% shall be placed in this category.
SPC 2	Buildings in compliance with the pre-1973 *California Building Standards Code* or other applicable standards, but not in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act. These buildings do not significantly jeopardize life, but may not be repairable or functional following strong ground motion. These buildings must be brought into compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, its regulations or its retrofit provisions by January 1, 2030, or be removed from acute care service. Where the office has performed a collapse probability assessment, buildings with Probability of Collapse less than or equal to 0.75% shall be placed in this category.
SPC 3	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, utilizing steel moment-resisting frames in regions of high seismicity as defined in Section 4.2.10 and constructed under a permit issued prior to October 25, 1994. These buildings may experience structural damage which does not significantly jeopardize life, but may not be repairable or functional following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained throughOSHPD. These buildings may be used to January 1, 2030, and beyond.
SPC 4	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, but may experience structural damage which may inhibit ability to provide services to the public following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used to January 1, 2030, and beyond.
SPC 5	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, and reasonably capable of providing services to the public following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used without restriction to January 1, 2030, and beyond.

1. Data collection on the structure and site conditions shall be performed in accordance with the appropriate Sections of Article 2 of these regulations. Depending upon the type of analysis to be performed, additional data regarding the as built condition and material properties may be required;
2. The Alternative Analysis shall be based on a site specific ground motion as specified in Section 3413A.1.2 of the 2007 *California Building Code* (CBC);
3. The analysis of the structure shall determine the distribution of strength and deformation demands produced by the design ground shaking and other seismic hazards. The analysis shall address seismic demands and capacities to resist these demands for all elements in the structure that either:
   - Are essential to the lateral stability of the structure (primary elements); or
   - Are essential to the vertical load-carrying integrity of the building.
4. The analysis procedure may consist of a linear or nonlinear analysis. The analytical methods and acceptance criteria shall conform to Section 3403A.2.3.4 of the 2007 CBC and nonlinear time history analysis procedure shall be reviewed and approved, in advance, by OSHPD.

ARTICLE 3
PROCEDURES FOR BUILDING SYSTEMS

3.0 Introduction. This article sets forth general requirements that apply to all buildings: load path, redundancy, configuration, adjacent buildings and the condition of the materials.

3.1 Load path. The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The load path is the most essential requirement for a building. There must be a lateral-force-resisting system that forms a load path between the foundation and all diaphragm levels and that ties all of the portions of the building together. The load path must be complete and sufficiently strong.

3.2 Redundancy. The structure will remain laterally stable after the failure of any single element.

Check whether stability of the structure depends on a single element. If the failure of a single element (member or connection) will result in loss of lateral stability, the element shall be checked for adequacy using an amplification factor of $C_d/2$, but not less than 1.5. *P*-delta effects shall be included in this check.

3.3 Configuration. Vertical irregularities are defined in terms of discontinuities of strength, stiffness, geometry and mass.

Horizontal irregularities involve the horizontal distribution of lateral forces to the resisting frames or shear walls. Irregularities in the shape of the diaphragm itself (i.e., diaphragms that are L-shaped or have notches) are covered in Article 7.

3.3.1 Weak story. Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check story strengths individually. Where a weak story exists, the resisting elements shall be checked; include *P*-delta effects and inelastic demand. To compensate for the concentration of inelastic action where the story strength of the weak story is less than 65 percent of the story above, amplify the design forces in the weak story by the factor $C_d/2$, but not less than 1.5. Conforming buildings which fail this check shall be placed in SPC 4.

3.3.2 Soft story. Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the stiffness of certain portions of the building. Where a soft story condition is indicated, the stiffness of the building shall be calculated story by story, in order to determine whether a story falls within the definition of a soft story. Where a soft story exists, the resisting elements shall be checked; include *P*-delta effects. For buildings more than 65 feet or five stories tall, a dynamic analysis shall be performed to compute the distribution of seismic forces.

3.3.3 Geometry. There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories).

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. Where geometric irregularities exist, a dynamic analysis shall be performed to compute the vertical distribution of seismic forces.

3.3.4 Mass. There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the distribution of mass in the building. The effective mass is the real mass consisting of the dead weight of the floor plus the actual weights of partitions and equipment. Where mass irregularities exist, a dynamic analysis shall be performed to compute the vertical distribution of seismic forces.

3.3.5 Vertical discontinuities. All shear walls, infilled walls and frames are continuous to the foundation.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The primary deficiency is in the strength of the columns that support the wall or frame. The secondary deficiency is in the strength of the connecting strut or diaphragm. Conforming buildings which fail these checks shall be placed in SPC 4.

Procedure for columns: Check the columns that support the upper vertical lateral load-resisting element for their capacity to support the gravity loads plus the overturning forces. The overturning forces shall be based on the design forces amplified by the factor $C_d/2$, but not less than 1.5, or on the capacity of the vertical lateral load-resisting element to resist lateral force if this is greater. The column check shall include *P*-delta effects.

Procedure for strut or diaphragm: Check the strut or diaphragm for its ability to transfer the load from the discontinuous element to the lower resisting element.

3.3.6 Torsion. 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 dimension.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. One deficiency is in the layout and the strengths and stiffness of the walls and frames of the lateral-force-resisting system. Another deficiency is in the strength of columns that are not part of the lateral-force-resisting system but are forced to undergo displacements due to the rotation of the diaphragm. 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 resisting elements. Calculate the maximum story drift (the average building drift plus the additional displacement due to torsion). Verify that all vertical load-carrying elements can maintain their load-carrying ability under the expected drifts. When checking columns, include *P*-delta effects and consider inelastic demand. Conforming buildings which fail this check shall be placed in SPC 4.

3.4 Adjacent buildings. There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated.

The deficiency is the distance between the buildings. Report the condition as a hazard. Where both buildings are designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated. Other conforming buildings which fail these checks shall be placed in SPC 4.

3.5 Deflection incompatibility. Column and beam assemblies that are not part of the lateral-force-resisting system (i.e., gravity load-resisting frames) are capable of accommodating imposed building drifts, including amplified drift caused by diaphragm deflections, without loss of vertical load-carrying capacity.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the ductility of the vertical load-carrying system. Calculate the expected drifts using the procedures in Section 2.4.4. Use net section properties for all reinforced concrete elements in the lateral-force-resisting system. Include the lateral displacements due to diaphragm deflections, using the diaphragm loading computed in Section 2.4.6. Evaluate the capacity of the nonlateral-force-resisting columns and beam assemblies to undergo the combined drift, considering moment-axial force interaction and column shear.

3.6 Short "captive" columns. There are no columns with height-to-depth ratios less than 75 percent of the nominal height-to-depth ratios of the typical columns at that level.

The deficiency is in the tendency of short captive columns to attract high shear forces because of their high stiffness relative to adjacent elements. Calculate the story drift, and determine the shear demand (V_e) in the short column caused by the drift $(V_e=2M/L)$. The ratio of V_e/V_n shall be less than one, where V_n is the column nominal shear capacity computed in accordance with ACI criteria. Conforming buildings which fail these checks shall be placed in SPC 4.

3.7 Evaluation of materials and conditions.

3.7.1 Deterioration of wood. None of the wood members shows signs of decay, shrinkage, splitting, fire damage or sagging, and none of the metal accessories is deteriorated, broken or loose.

The deficiency is in the capacity of the deteriorated elements. Determine the cause and extent of damage. Identify the lateral-force-resisting system and determine the consequences of the damage to the system. The system shall be judged adequate if it can perform with the damaged elements. Check the structural systems with appropriate reductions in member properties.

3.7.2 Overdriven nails. There is no evidence of overdriven nails in the shear walls or diaphragms.

The deficiency is in the capacity of the fasteners. Check the wall demand and capacity, using reduced strength due to overdriven fasteners.

3.7.3 Deterioration of steel. There is no significant visible rusting, corrosion or other deterioration in any of the steel elements in the vertical- or lateral-force-resisting systems.

The deficiency is the reduction in cross-section of the elements. Check the structural systems with appropriate reductions in member properties. See Article 4 for inspection requirements for welded steel moment-resisting frame structures.

3.7.4 Deterioration of concrete. There is no visible deterioration of concrete or reinforcing steel in any of the frame elements.

The deficiency is the reduction in member properties. Check the structural systems with appropriate reductions in member capacities.

3.7.5 Post-tensioning anchors. There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors have not been used.

The deficiency is the reduced area of the prestress strands and, with coil anchors, the ability of the anchorage to maintain its grip under cyclic loading. Inspect a sample of the concrete in the area of the anchorage to determine its condition. Determine the cause and extent of the deterioration. Consider the effects of anchorage failure on the vertical and lateral load-carrying capacity of the structure.

3.7.6 Concrete wall cracks. All diagonal cracks in the wall elements are 1.0 mm or less in width, are in isolated locations and do not form an X pattern.

The deficiency is the reduced capacity of the wall. Determine the cause and extent of the cracking. Check the structural systems with reduced wall capacity.

3.7.7 Cracks in boundary columns. There are no diagonal cracks wider than 1.0 mm in concrete columns that encase the masonry infills.

The deficiency is the reduced capacity of the wall. Evaluate the wall with limited capacity assigned to the deteriorated elements. Determine the cause and extent of the damage.

3.7.8 Precast concrete walls. There is no significant visible deterioration of concrete or reinforcing steel or evidence of distress, especially at the connections.

The deficiency is in the strength of the connections. Determine the cause and extent of distress and check the structural systems with appropriate reductions in capacity.

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

The deficiency is in the strength of the wall. Check the adequacy of the walls with the strength determined by tests. This evaluation statement also applies to masonry veneers present on the exterior or interior walls of the building.

3.7.10 Masonry units. There is no visible deterioration of large areas of masonry units.

The deficiency is in the strength of the units. Determine the cause and extent of deterioration and use reduced capacity in determining the adequacy of the units.

3.7.11 Cracks in infill walls. There are no diagonal cracks in the infilled walls that extend throughout a panel or are greater than 1.0 mm wide.

The deficiency is the reduced capacity of the wall. Determine the cause and extent of the cracking. If appropriate, check the structural systems with reduced wall capacity.

ARTICLE 4 PROCEDURES FOR MOMENT-RESISTING SYSTEMS

4.0 Introduction. Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements. Moment frames may be classified as special, intermediate and ordinary frames.

For evaluations using these regulations, it is not necessary to determine the type of frame in the building. The issues are addressed by appropriate acceptance criteria in the specified procedures. For determination of element capacities, see Article 2, Section 2.4.9.

4.1 Frames with infill walls.

4.1.1 Interfering walls. All infill walls placed in moment frames are isolated from structural elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is an inappropriate connection of the wall to the frame. Evaluate the relative strength and stiffness of the walls and frames, considering the nature and size of the joint or connection between the wall and the frame. If the strength of the walls is not commensurate with the stiffness, the building should be treated as Type 7 or Type 10 (Article 2, Section 2.2.3 "Common Building Types"), a frame with infill walls. If the infill walls do not extend the full story height and are not properly isolated from the frame columns, evaluate the column shear demand and capacity, based on a column height equal to the clear distance from the top of the wall to the bottom of the slab or beam above, amplifying the design forces in the short column by $C_d/2$, but not less than 1.5. The shear demand need not exceed the shear capacity corresponding to flexural capacity of the column, based on a column height equal to the clear distance from the top of the wall to the bottom of the slab or beam above.

4.2 Steel moment frames. Welded steel moment frames may be subject to detailed frame joint evaluation requirements, as outlined in this section. The purpose of this joint evaluation is to determine if the building has experienced joint damage in strong ground shaking.

4.2.0.1 Preliminary screening. All welded steel moment frame structures shall undergo a detailed frame joint evaluation if the building is located at a site that has experienced the following:

1. An earthquake of magnitude greater than or equal to 6.5 that produced ground motion in excess of 0.20 g; or
2. An earthquake that generated ground motion in excess of 0.30 g.

The ground motion estimates shall be based on actual instrumental recordings in the vicinity of the building. When such ground motion records are not available, ground motion estimates may be based on empirical or analytical techniques. All ground motion estimates shall reflect the site-specific soil conditions.

4.2.0.2 Additional indicators. A detailed frame joint evaluation of the building shall be performed if any of the following apply:

1. Significant structural damage is observed in one or more welded steel moment frame structures located within 1 km of the building on sites with similar, or more firm, soil properties;
2. An earthquake having a magnitude of 6.5 or greater, where the structure is located within 5 km of the trace of a surface rupture or within the vertical projection of the rupture area when no surface rupture has occurred;
3. Significant architectural or structural damage has been observed in the building following an earthquake; or

4. Entry to the building has been limited by the building official because of earthquake damage, regardless of the type or nature of the damage.

4.2.0.3 Connection inspections. Detailed frame joint evaluations shall be performed in accordance with the procedures in the *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures,* FEMA 267, August 1995.

4.2.1 Drift check. The building satisfies the Quick Check of the frame drift.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check drift using the procedures in Section 2.4.7.1 against the prescribed limit. If the drift exceeds the limiting drift at any story level, the structure shall be evaluated with full-frame analysis using the anticipated distribution of lateral forces to the moment-resisting frames and including *P*-delta effects. Check the other statements using the demand from this analysis.

4.2.2 Compact members. All moment-frame elements meet the compact section requirements of the basic AISC documents.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the member capacities. Check member capacities, using member demands obtained from a frame analysis. Calculate member capacities using appropriate criteria for noncompact sections. Check the member capacities using appropriate *R* values (e.g., noncompact members require use of the *R* value for ordinary frames).

4.2.3 Beam penetrations. All openings in frame-beam webs have a depth less than one-fourth of the beam depth and are located in the center half of the beams.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the shear capacity of the beam. Check that the shear capacity of the beam is sufficient to develop the flexural plastic hinge. If the shear capacity is insufficient to develop the flexural capacity of the member, use the *R* value for ordinary frames.

4.2.4 Moment connections. All beam-column connections in the lateral-force-resisting moment frame have full-penetration flange welds and a bolted or welded web connection.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection. Check the connection on the basis of its strength. Check the member capacities using appropriate *R* values. Connections that do not develop the flexural capacity of the member require use of the *R* value for ordinary frames.

4.2.5 Column splices. All column splice details of the moment-resisting frames include connection of both flanges and the web.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the bolts or welds in the connection. Check the adequacy of the splice connection for all gravity and seismic loads. Amplify the seismic load for partial-penetration welded splices by the factor $C_d/2$.

4.2.6 Joint webs. All web thicknesses within joints of moment-resisting frames meet AISC criteria for web shear.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the web. Calculate the joint shear capacity using formulas given in the AISC provisions and compare it to the demand from an equivalent lateral force analysis or the average column shear, V_c, calculated for the Quick Check for drift.

4.2.7 Girder flange continuity plates. There are girder flange continuity plates at joints.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the joint. Check joints without such plates using AISC provisions, using the *R* value for ordinary frames.

4.2.8 Strong column/weak beam. At least one half of the joints in each story are strong column/weak beam (33 percent on every line of moment frame). Roof joints need not be considered.

The deficiency is excessive ductility demand and displacement in a single story. Compare beam and column moment capacities, including the effect of axial force. The evaluator may consider this condition mitigated if the joints in the building meet the provisions of Section 2710(g)5 of the 1992 edition of Part 2, Title 24. Conforming buildings which do not meet those provisions shall be placed in SPC 4.

4.2.9 Out-of-plane bracing. Beam-column joints are braced out-of-plane.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the stability of the beam-column joint. Verify the joint bracing by visual observation.

4.2.10 Pre-Northridge earthquake welded moment frame joints. Welded steel moment frame beam-column joints are designed and constructed in accordance with recommendations in FEMA 267, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures,* August 1995.

For buildings constructed under permit issued after October 25, 1994, the evaluator may consider this condition as mitigated. The deficiency is in the ductility of the beam-column joint. The following procedures shall be used for categorizing buildings with welded steel moment frame joints:

Procedure for conforming buildings: Conforming buildings located in Seismic Zone 4 of 1995 *California Building Code* (CBC) or later version of the CBC, within a zone designated as being potentially subject to near field effects in strong ground shaking, shall be placed in SPC 3.

All other conforming buildings shall be placed in SPC 4.

Procedure for nonconforming buildings: Nonconforming buildings shall be placed in SPC 2.

4.3 Concrete moment frames. The details covered in evaluation statements in Sections 4.3.4 through 4.3.14 will be found in frames that have been designed and detailed for ductile behavior. If any one detail is not present, the frames are not considered to meet life-safety goals, and nonconforming buildings shall be placed in SPC 1. For conforming buildings, see the appropriate evaluation statement. For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the building may assume "true" responses to all evaluation statements in this section.

4.3.1 Shearing stress check. The building satisfies the Quick Check of the average shearing stress in the columns.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Perform a quick estimation of the average shearing stress in the columns according to the procedure specified in Section 2.4.7.2. If the average column shear stress is greater than 60 psi, a more detailed evaluation of the structure shall be performed. This evaluation shall employ a more accurate estimation of the level and distribution of the lateral loads; use the procedures outlined in Section 2.4.

4.3.2 Drift check. The building satisfies the Quick Check of story drift.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check drift using the procedures in Section 2.4.7.1 against the prescribed limit. If the drift exceeds the limiting drift at any story level, the structure shall be evaluated with full-frame analysis using the anticipated distribution of lateral forces to the moment-resisting frames and including *P*-delta effects as found in Section 2.4.1. Check the other statements using the demand from this analysis.

4.3.3 Prestressed frame elements. The lateral-load-resisting frames do not include any prestressed or post-tensioned elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the frames during inelastic straining. Check the capacity of the members and joints using all of the mild steel reinforcing that is available and bonded prestressing when appropriate. The *R* value used for evaluation shall reflect the ductility and damping of the system. Where better information is not available, multiply the *R* value selected on the basis of mild reinforcement by 0.75 to account for the effect of prestressing.

4.3.4 Joint eccentricity. There are no eccentricities larger than 20 percent of the smallest column plan dimension between girder and column centerlines.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the frame, either the members or the joints or both. Evaluate the frames considering the additional shear stresses caused by the joint torsion.

4.3.5 No shear failures. The shear capacity of frame members is greater than the moment capacity.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is inadequate shear capacity in the columns or beams. Compare V_e with the member shear capacity, ϕV_n, calculated in accordance with ACI 318 Appendix. The ratio $V_e/\phi V_n$ shall be less than or equal to 1.0.

4.3.6 Strong column/weak beam. The moment capacity of the columns is greater than that of the beams.

The deficiency is in column capacity. Compare the sum of the beam moment capacities to that of the column capacities. Include the participation of the slab in the beam capacities. The moment capacity to be compared is the plastic moment, M_{pr}. The ratio of the sum of the M_{pr} for the columns to the sum of the M_{pr} for the beams is required to be not less than 1.2. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.7 Stirrup and tie hooks. The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more.

The deficiency is in the shear resistance and confinement of the member. Determine if beam stirrups and column ties are appropriately anchored into member cores with hooks of 135 degrees or more. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.8 Column-tie spacing. Frame columns have ties spaced at $d/4$ or less throughout their length and at 8 d_b, or less at all potential plastic hinge regions.

The deficiency is in the shear capacity of the column. Report this condition as a deficiency. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.9 Column-bar splices. All column bar lap splice lengths are greater than 35 d_b, long and are enclosed by ties spaced at 8 d_b, or less.

The deficiency is in the strength and ductility of the column. Compare the splice length provided with that required by Sections 12.2 and 12.15 of the ACI 318 provisions. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.10 Beam bars. At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25 percent of the steel provided at the joints for either positive or negative moment is continuous throughout the members.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam. Determine if the required beam bars are present. For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary.

4.3.11 Beam-bar splices. The lap splices for longitudinal beam reinforcing are located within the center half of the member lengths and not in the vicinity of potential plastic hinges.

The deficiency is in the strength and ductility of the beam. Determine if the beam bar splices are detailed and located such that the yield capacity of the beam can be developed. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.12 Stirrup spacing. All beams have stirrups spaced at $d/2$ or less throughout their length and at 8 d_b, or less at potential hinge locations.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam. Determine if the stirrups meet the specified spacing requirements, such that the yield capacity of the beam can be developed.

4.3.13 Beam truss bars. Bent-up longitudinal steel is not used for shear reinforcement.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam. Determine if bent-up shear reinforcement is present. If present, check the shear capacity of the element ignoring the effects of the bent-up longitudinal bars.

4.3.14 Joint reinforcing. Column ties extend at their typical spacing through all beam-column joints at exterior columns.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam-column joint. Calculate the joint capacity, V_e, and the joint shear, V_j. The joint shear is calculated at a horizontal section at mid-height of the joint. The horizontal shear at the critical section is obtained from summation of horizontal forces in a free-body diagram of the upper half of the joint as $V_j = (T_l + T_r) - V_e$ where T_l and T_r, the forces in the flexural tensile reinforcement in the beams on the left and right sides of the joint, respectively, are calculated assuming a steel stress equal to 1.25 f_y. See Figure 4.3.14 for computation of V_e. The ratio V_j/V_e shall be less than or equal to 1. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.15 Flat Slab frames. The system is not a frame consisting of columns and a flat slab/plate without beams.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. Perform a detailed analysis, or assign the building to SPC 1.

4.4 Precast concrete moment frames.

4.4.1 Precast frames. The lateral loads are not resisted by precast concrete frame elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connections. Check the adequacy of the precast frames. Where lateral movement will cause strength capacities to be first exceeded at connections, use $R = C_d = 1.5$ unless there is information on connection behavior that justifies higher values. Where all yielding occurs within members, use the R-value for the appropriate cast-in-place frame.

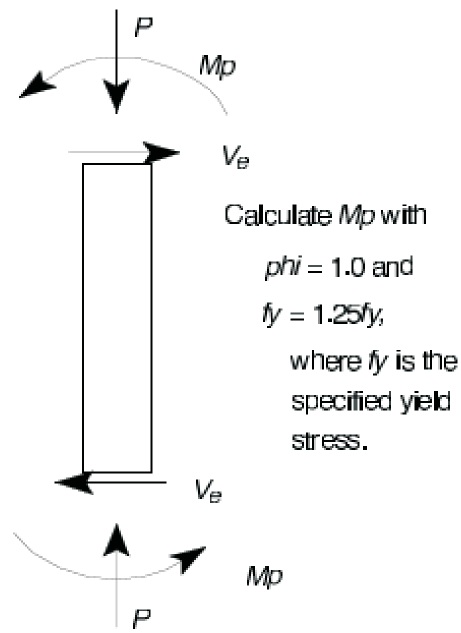


FIGURE 4.3.14
M_{pr} and V_e

4.4.2 Precast connections. For buildings with concrete shear walls, the connection between precast frame elements such as chords, ties and collectors in the lateral-force-resisting system can develop the capacity of the connected members.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connections. Analyze the connections. Determine where connection failures would be brittle (e.g., pull-out of an embedded item would occur before yield of a mild steel element). Analyze structure for stability assuming that these brittle connections have failed or are not capable of transmitting forces, or check such connections for seismic force amplified by factor $C_d/2$, but not less than 1.5. For shear capacity, refer to Section 4.3. For flexure, find the path of forces from the element through the connection into the other element.

4.5 Frames not part of the lateral-force-resisting system. This section deals with frames that were not designed to be part of the lateral-force-resisting system. These are basic structural frames of steel or concrete that are designed for gravity loads with shear walls, bracing or moment frames providing the resistance to lateral forces.

If the primary lateral-force-resisting system consists of concrete walls (infilled in steel frames or monolithic in concrete frames), the building shall be treated as a concrete shear wall building (Type 6) with the frame columns as boundary elements. If the walls are masonry infills, the frames shall be treated as steel or concrete frames with infill walls of masonry (Type 7 or 10). Buildings with steel braces shall be treated as braced frame systems (Type 4). The principal deficiency identified in this section is loss of vertical-load-carrying capacity due to excessive deformations.

The analysis must include the deformations imposed by the infill walls, and the consequences of the failure of such walls.

4.5.1 Complete frames. The steel or concrete frames form a complete vertical load-carrying system.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check the shear walls or braced frames, including the effects of all dead and live loads, and note that the *R* values for buildings without a complete vertical load-carrying space frame are different from those for complete frame buildings. For wall systems, the frame is considered incomplete if the beams end at the edge of a shear wall that has no boundary columns or, if there are such columns, the beams do not continue across in the plane of the wall. For chevron-braced frame systems, the frame is considered incomplete if the beam in the brace frame cannot carry the design dead and live loads without the presence of the braces.

ARTICLE 5
PROCEDURES FOR SHEAR WALLS

5.0 Introduction. Shear walls have two aspects: carrying in-plane shear when the earthquake direction under consideration is parallel to the wall and resisting out-of-plane forces when the earthquake direction under consideration is perpendicular to the wall. The in-plane effects are covered in this article. Out-of-plane effects are covered in Article 8. All walls not structurally isolated are assumed to act as shear walls that will participate in resisting lateral forces up to their capacity.

5.1 Concrete shear walls.

5.1.1 Shearing stress check. The building satisfies the Quick Check of the shearing stress in the shear walls.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Generate the lateral loads using the Quick Check procedure of Section 2.4.7.3. If v_{avg} is greater than 50 psi (or square root of f'_c if f'_c is known), a more detailed evaluation of the structure shall be performed. This evaluation shall employ a more accurate estimation of the level and distribution of the lateral loads, using the analysis procedures in Article 2.

5.1.2 Overturning. All shear walls have h_w/l_w ratios less than 4 to 1.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the required resistance to overturning moments. Calculate the resistance to the required overturning moments. The overturning resistance shall include the resistance contributed by wall flanges, friction on piling, earth over foundations, and floor and roof weights supported by the wall. The calculated resistance shall be greater than 0.75 times the base moment of the shear wall. The overturning resistance moment may be taken as the righting moment about an edge of the footing or the wall flexural capacity, whichever is less.

5.1.3 Coupling beams. The stirrups in all coupling beams are spaced at *d/2* or less and are anchored into the core with hooks of 135 degrees or more.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the coupling beams. Assume that the beams yield. Calculate their end-moment capacity based either on flexural yield or shear capacity, whichever is lower. The coupling beam moment capacity should include the contribution of a reasonable portion of the adjacent floor slab reinforcement when this reinforcement is in tension. Analyze the walls as independent walls with these restoring moments or shears helping to stabilize the walls. Check the stability of the wall and the stresses in the vertical boundaries. Conforming buildings which fail this check shall be placed in SPC 4, and no calculations are necessary.

5.1.4 Column splices. Steel column splice details in shear wall boundary elements can develop the tensile strength of the column.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the splice in the boundary column. Determine the maximum tensile column load in each case and verify the adequacy of the splice to resist this load, including gravity loads. Check the adequacy of the splice connection for all gravity and seismic loads. Amplify the seismic load for partial-penetration welded splices by the factor $C_d/2$, but not less than 1.5, when the seismic load produces tension at the splice.

5.1.5 Wall Connections. There is positive connection between the shear walls and the steel beams and columns.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the adequacy of the connections between the shear wall and the beams and columns that are its boundary elements. Calculate the effective overturning demand for the walls and check the adequacy of the shear transfer to the steel elements. A value for shear friction between steel and concrete shall be included only if the steel element is completely encased with reinforced concrete.

5.1.6 Confinement reinforcing. For shear walls with h_w/l_w greater than 2.0, the boundary elements are confined with spirals or ties with spacing less then $8d_b$.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the ductility of the vertical boundary elements that are required to resist large axial forces. Check the need for boundary elements, per ACI 318. Where boundary elements are required but not provided, amplify the seismic forces for the entire structure by the factor 1.25 (and use $0.8C_d$ for drift calculation). Conforming buildings which fail this evaluation statement shall be placed in SPC 4, and no calculations are necessary.

5.1.7 Reinforcing steel. The total reinforcing steel for concrete walls is greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes and the maximum spacing of reinforcing steel is 18 inches.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the quantity of reinforcing in the wall. Calculate the capacity of the walls with the reinforcing that is provided, but amplify the seismic forces by the factor 1.25 (and use $0.8C_d$ for drift calculation). Where the reinforcing in the wall is less than 0.0015 times the gross area of the wall along the longitudinal or transverse axis, or if the reinforcing steel spacing exceeds 18 inches, the contribution of the wall to lateral strength and stiffness of the building shall be ignored and, if it is a bearing wall, the building shall be placed in SPC 1.

5.1.8 Reinforcing at openings. There is special wall reinforcement around all openings.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the reinforcing in the piers and spandrels. Determine the capacity of the spandrels and piers considering all available reinforcing steel that crosses the critical sections.

5.2 Precast concrete shear walls. Shear walls of precast concrete are in segments that are tied together, but the connections may be of a brittle type. Connections adequate for design level forces may not be capable of developing the yield level capacity of the panels. The effects of the precast panel connections on the other evaluation statements concerned with wall elements shall be considered. The deficiency is in the quality and ductility of the connections.

5.2.1 Panel-to-panel connections. Adjacent wall panels are not connected by welded steel inserts.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the inserts. Check the welded inserts. Determine where connection failures would be brittle (e.g., pull-out of an embedded item would occur before yield of a mild steel element). Analyze structure for stability assuming that these brittle connections have failed or are not capable of transmitting forces or check such connections for seismic force amplified by the factor $C_d/2$, but not less than 1.5.

5.2.2 Wall openings. Openings constitute less than 75 percent of the length of any perimeter wall with the wall piers having h_w/l_w ratios of less than 2.0.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency may be in the strength of the panel connections or may be that the reinforced concrete elements actually behave like a moment frame and should be evaluated as such. Check the elements in the precast shear wall system. When large open areas exist, check the transfer of shear between the diaphragm and the wall. Compare the lateral displacements of the wall due to shear and flexure. If more than 50 percent of the total lateral displacement is due to flexure, or if the width of the wall piers is less than five times the thickness, analyze the wall as a moment frame.

5.2.3 Collectors. Wall elements with openings larger than a typical panel at a building corner are connected to the remainder of the wall with collector reinforcing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the configuration of the wall or the diaphragm. Find an adequately strong path of forces. If none is found, report this as a deficiency.

5.3 Reinforced masonry shear walls.

5.3.1 Shearing stress check. The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Generate the lateral loads using the Quick Check procedure of Section 2.4.7.3. If v_{avg} is greater than 15 psi, a more detailed evaluation of the structure shall be performed. This evaluation shall employ a more accurate estimation of the level and distribution of the lateral loads, using the analysis procedures in Article 2.

5.3.2 Reinforcing. The total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions, the spacing of reinforcing steel is less than 48 inches and all vertical bars extend to the top of the walls.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. If the quantity of wall reinforcing is less than the specified amounts, report this condition as a deficiency.

5.3.3 Reinforcing at openings. All wall openings that interrupt rebar have trim reinforcing on all sides.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the lack of reinforcing at the end of wall elements adjacent to openings and at the corners of walls. Check the wall using only the length of piers between reinforcing steel.

5.4 Unreinforced masonry shear walls. Unreinforced masonry bearing wall buildings are automatically classified as SPC 1. The following provisions apply to unreinforced masonry shear wall structures that also possess a complete vertical load-carrying space frame.

5.4.1 Shearing stress check.

The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls.

Generate the lateral loads using the Quick Check procedure of Section 2.4.7.3. The allowable stress (on the gross area) for solid brick masonry is 10 psi; for hollow unit masonry, 6 psi; and for grouted block masonry, 12.5 psi. If v_{avg} is greater than the allowable stress, an Alternative Analysis of the structure shall be performed, or the building shall be placed in SPC 1.

5.4.2 Masonry lay-up.

Filled collar joints of multiwythe masonry walls have negligible voids.

The deficiency is in the lay-up of the wall that left voids between the wythes. Investigate the lay-up. This can be done when masonry units are removed for strength tests. If voids are present, report this condition as a deficiency.

5.5 Unreinforced masonry infill walls in frames.

5.5.1 Proportions.

The height/thickness ratio of the wall panels is as follows:

One-story building $h_w/t < 14$

Multistory building

Top story $h_w/t < 9$

Other stories $h_w/t < 20$

The deficiency is in the out-of-plane strength of the wall. Check the out-of-plane demand using the procedure for parts and portions of a building given in Section 2.4.6.

5.5.2 Solid walls. The infill walls are not of cavity construction.

The deficiency is in the out-of-plane strength of the wall. If infill walls are of cavity construction, report this as a deficiency.

5.5.3 Infill walls. The infill walls are continuous to the soffits of the frame beams.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the columns. Check the shear capacity of the columns to develop opposing yield moments at top and bottom of the short free height or to resist required force amplified by the factor $C_d/2$, but not less than 1.5.

5.5.4 Wall connections. All infill panels are constructed to encompass the frames around their entire perimeter.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the connection of the infill panel to the frame. Determine the panel edge condition from available drawings or from field investigation. If the panels are not properly connected to the frame, report this condition as a deficiency.

5.6 Walls in wood frame buildings.

5.6.1 Shearing stress check. The building satisfies the Quick Check of the shearing stress in wood shear walls.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Generate the lateral loads using the Quick Check procedure of Section 2.4.7.3 and compare to 400 pounds per foot of plywood wall or 50 pounds per foot of walls composed of gypsum board or other materials. If v_{avg} is greater than these values, a more detailed evaluation of the structure shall be performed. This evaluation shall employ a more accurate estimation of the level and distribution of the lateral loads using the analysis procedures in Article 2.

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

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the lateral-force-resisting system. Check the ability of the walls and diaphragms to control, through torsional capacity, displacements at walls with large openings. Check that the diaphragm is a complete system with chords and collectors provided to deliver the lateral loads as required.

5.6.3 Wall requirements. All walls supporting tributary area of 24 to 100 square feet per foot of wall are plywood sheathed with proper nailing or rod braced and have a height-to-depth *(H/D)* ratio of 1 to 1 or less or have properly detailed and constructed hold-downs.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the wall and/or in hold-downs to resist overturning forces. Check the walls using floor areas tributary to the walls. Check all portions of the load path to ensure proper force transfer.

5.6.4 Cripple walls. All exterior cripple walls below the first floor level are braced to the foundation with shear elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the shear strength of the cripple walls. Check all exterior cripple walls below the first floor level to ensure that they are braced to the foundation with shear elements.

5.6.5 Narrow shear walls. Narrow wood shear walls with an aspect ratio greater than 2 to 1 do not resist forces developed in the building.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the narrow walls. Determine the shear capacity of the wall and related overturning demand. This shear capacity and related overturning must be transferred to the foundation within allowable stresses.

5.6.6 Stucco (exterior plaster) shear walls. Multistory buildings do not rely on exterior stucco walls as the primary lateral-force-resisting system.

The deficiency is in the strength of the stucco walls. Inspect stucco-clad buildings to determine if there is a lateral system such as plywood or diagonal sheathing at all but the top floor. Where exterior plaster is present, verify that the wire reinforcing is attached directly to the wall framing and the wire is completely embedded into the plaster material. Conforming buildings which fail this check shall be placed into SPC 4.

5.6.7 Plaster or gypsum wallboard shear walls. Interior plaster or gypsum wallboard is not being used for shear walls in buildings over one story in height.

The deficiency is in the strength of the walls. Determine if there is a lateral system such as plywood or diagonal sheathing at all but the top floor. Multistory buildings shall not rely on interior plaster or gypsum wallboard walls as the primary lateral-force-resisting system. Conforming buildings which fail this check shall be placed into SPC 4.

ARTICLE 6
PROCEDURES FOR BRACED FRAMES

6.0 Introduction. Braced frames develop their resistance to lateral forces by the bracing action of diagonal members. The braces induce forces in the associated beams and columns so that all work together like a truss with all members subjected to stresses that are primarily axial.

A **Concentrically braced frame** has minor eccentricities in the joints of the frame that are accounted for in the design.

An **Eccentrically braced frame** has elements that are strictly controlled to combine a stiffening effect due to the diagonal braces with yielding in the link beams. Eccentrically braced frames are present only in conforming buildings.

6.1 Concentrically braced frames.

6.1.1 Stress check. The building satisfies the Quick Check of the stress in the diagonals.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Calculate the average axial stress in the diagonals using the procedures of Section 2.4.7.4. Increase the calculated stress to account for torsion, based on the amount of torsion (Section 3.3.6) present and the distance between braced frames. If the average stress exceeds 30 ksi, an accurate analysis of the stresses on the bracing elements shall be performed.

6.1.2 Stiffness of diagonals. All diagonal elements required to carry compression have *Kl/r* ratios less than 120.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the stiffness of the diagonals. Check the bracing elements, amplifying the seismic force by the factor 1.25.

6.1.3 Tension-only braces. Tension-only braces are not used as the primary diagonal bracing elements in structures over two stories in height.

The deficiency is in the strength of the braces. Check the braces. If they are tension-only, and the building is over two stories in height, place the building in SPC 1. Tension-only bracing of small penthouse structures may be reviewed using the procedures in Section 2.4.6. Conforming buildings which fail this check shall be placed in SPC 4.

6.1.4 Chevron bracing. The bracing system does not include chevron-, V-, or K-braced bays.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check all elements in the braced frames. For chevron- and V-braced frames, the beam shall be a single element that can carry the gravity loads without the intermediate support of the braces. Check the adequacy of the beam for the seismic forces amplified by $C_d/2$, but not less than 1.5. Consider the effect of buckling of a leg of chevron-bracing or V-bracing, including the continuity, strength, and bracing of the beams and the ability of the connection to permit buckling of the brace while not destroying the capacity for repeated cycles of loading. If K-bracing is used in buildings over two stories, amplify the seismic forces in the bracing and columns by the factor $C_d/2$, but not less than 1.5.

6.1.5 Concentric joints. All the diagonal braces frame into the beam-column joints concentrically.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the joints. Evaluate the consequence of the eccentricity on the member required to resist it. Evaluate the shear, bending and axial force requirements at the locations of eccentricities.

6.1.6 Connection strength. All the brace connections are able to develop the yield capacity of the diagonals.

The deficiency is in the strength of the connections. Check the connection strength. Use a demand value that develops the tensile capacity of the brace or is 1.25 times the required seismic force. If connections in a conforming building cannot develop the yield capacity of the brace and do not meet the requirements of Part 2, Title 24, Section 2211A.9.3 of 1995 *California Building Code* (CBC) or equivalent provision in later version of the CBC, the building shall be placed in SPC 4.

6.1.7 Column splices. All column splice details of the braced frames can develop the column yield capacity.

The deficiency is in the strength of the splice. Calculate the adequacy of the splice connection for all expected forces including gravity loads. Amplify the seismic load for partial penetration welded splices by the factor $C_d/2$ when the seismic load produces tension at the splice. If the column splice details in a conforming building cannot develop the yield capacity of the column and do not meet the requirements of Part 2, Title 24, Section 2211A.9.5 of 1995 *California Building Code* (CBC) or equivalent provision in later version of the CBC, the building shall be placed in SPC 4.

6.1.8 Concrete braced frames. None of the braces in the framing system are of reinforced concrete construction.

The deficiency is in the ductility of the braced frame. Report this condition as a deficiency, and place nonconforming buildings in SPC 1. Place conforming buildings in SPC 4.

6.2 Eccentrically braced frames.

6.2.1 Link beam location. The link beams are not connected to the columns.

The deficiency is in the ductility of the link beam-column connection. Report this condition and place the building in SPC 4.

ARTICLE 7
PROCEDURES FOR DIAPHRAGMS

7.0 Introduction. The diaphragm is the horizontal subsystem that distributes lateral load to the vertical subsystems (walls and frames) and that provides lateral support for walls and parapets.

7.1 Diaphragms. Diaphragms are treated as horizontal beams. The floor (or roof), which is analogous to the web of a wide-flange beam, is assumed to carry the shear; the edge of the floor (or roof) or a spandrel, which is analogous to the flange, is assumed to carry the flexural stress.

7.1.1 Plan irregularities. There is significant tensile capacity at reentrant corners or other locations of plan irregularities.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm in the vicinity of corners. Evaluate the chord/collector requirements at the reentrant corners and other locations of plan irregularities by applying the maximum of the diaphragm force and the calculated story force to a model of the isolated diaphragm. All elements that can contribute to the tensile capacity at the reentrant corner may be included with appropriate consideration given to gravity load stresses. Conforming buildings which fail this check shall be placed in SPC 4.

7.1.2 Cross ties. There are continuous cross ties between diaphragm chords.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the adequacy of the path for wall anchorage forces into the diaphragm. A cross tie is a beam or girder that spans the width of the diaphragm, accumulates the wall loads and transfers them, over the full depth of the diaphragms, into the next bay and on to the nearest shear wall or frame. Calculate the wall anchorage forces according to Section 2.4.5, and check that these forces can be developed, element by element, in the diaphragm.

7.1.3 Reinforcing at openings. There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm in the vicinity of the openings. Check the adequacy of the diaphragm to transfer stresses around the opening.

7.1.4 Openings at shear walls. Diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length, and the available length appears sufficient.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the length of diaphragm needed to transfer shear to the wall or frame and to provide lateral support for the wall or frame.

Procedure for diaphragm shear: Verify that there is a path of forces and sufficient strength to deliver the diaphragm shear to the shear wall. The diaphragm shear is the demand.

Procedure for lateral support of the wall: Treat the wall as a portion of the building using F_p as the demand.

7.1.5 Openings at braced frames. Diaphragm openings immediately adjacent to the braced frames extend less than 25 percent of the length of the bracing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is similar to that described above for openings at shear walls.

Procedure for diaphragm shear: Verify that there is a path of forces and sufficient strength to deliver the diaphragm shear to the braced frame. The diaphragm shear is the seismic demand.

Procedure for lateral support of the frame: Treat the frame as a portion of the building using F_p as the demand.

7.1.6 Openings at exterior masonry shear walls. Diaphragm openings immediately adjacent to exterior masonry walls are no more than 8 feet long.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is similar to that described above for openings at shear walls.

Procedure for diaphragm shear: Verify that there is a path of forces and sufficient strength to deliver the diaphragm shear to the shear wall. The diaphragm shear is the demand.

Procedure for lateral support of the wall: Treat the wall as a portion of the building using F_p as the demand.

7.2 Wood diaphragms.

7.2.1 Sheathing. None of the diaphragms consist of straight sheathing or have a span/depth ratio greater than 2 to 1.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm. Analyze the wood diaphragm using the procedure given in Chapter 9 of the 1994 *NEHRP Recommended Provisions.*

7.2.2 Spans. All diaphragms with spans greater than 24 feet have plywood or diagonal sheathing. Structures in Building Type 2 may have rod-braced systems.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and stiffness of the diaphragm. Evaluate the diaphragm stresses using the procedure given in Chapter 9 of the 1994 *NEHRP Recommended Provisions.* Also evaluate the deflections. A maximum displacement of 3 inches shall be acceptable. For horizontal bracing systems, see Section 7.5.

7.2.3 Unblocked diaphragms. Unblocked wood panel diaphragms consist of horizontal spans of less than 40 feet and have span/depth ratios less than or equal to 3 to 1.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm. Analyze the diaphragm using the 1994 *NEHRP Recommended Provisions* requirements for unblocked diaphragms.

7.2.4 Span/depth ratio. If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the stiffness of the diaphragm. Analyze the wood diaphragm using the procedures given in Chapter 9 of the 1994 *NEHRP Recommended Provisions.*

7.2.5 Diaphragm continuity. None of the diaphragms are composed of split-level floors or, in wood commercial or industrial buildings, have expansion joints.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm. Evaluate the building with proper recognition of the effects of the discontinuities.

7.2.6 Chord continuity. All chord elements are continuous, regardless of changes in roof elevation.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is the lack of a chord. Report the lack of a chord as a deficiency.

7.3 Metal deck diaphragms. Allowable values of metal deck diaphragms may be obtained from the manufacturer's approved data. The evaluator shall consider conditions that can weaken the diaphragm (i.e., troughs, gutters and recesses that have the effect of reducing the system to the bare deck or of creating a joint).

7.3.1 Deck topping. All metal deck roofs have a reinforced concrete topping slab.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm. Evaluate the bare metal deck diaphragm using the procedure given in the 1994 *NEHRP Recommended Provisions* requirements.

7.3.2 Untopped diaphragms. Untopped metal deck diaphragms consist of horizontal spans of less than 40 feet and have span/depth ratios less than or equal to 3 to 1.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the diaphragm. Analyze the diaphragm using the procedure given in the 1994 *NEHRP Recommended Provisions* requirements.

7.4 Precast concrete diaphragms. Evaluation of precast concrete diaphragms and the connections between precast elements shall consider eccentricities, adequacy of welds and length of embedded bars. If a topping slab is provided, it shall be assumed to resist all of the shear.

7.4.1 Topping slab. Precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab.

The deficiency is in the ability to transfer shear from one element to another. Check the slab element interconnection and check the lateral load capacity of the vertical elements that resist horizontal force. Where the capacity of the diaphragm is less than 150 percent of the sum of the load capacities of the vertical elements and where connections can allow the diaphragm to fail in a brittle manner, the R values used in computing the seismic demand shall be consistent with those for brittle systems (not to exceed $R = 2$). Conforming buildings without a reinforced concrete topping slab shall be placed in SPC 4.

7.4.2 Continuity of topping slab. The topping slab continues uninterrupted through the interior walls and into the exterior walls or is provided with dowels with a total area equal to the topping slab reinforcing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is the abrupt loss of strength where the topping slab is interrupted. Evaluate the tension and shear demand due to diaphragm forces, including collector requirements, perpendicular-to-wall loads, or chord forces at re-entrant corners.

7.5 Horizontal bracing. Horizontal bracing forms a complete system of adequate capacity.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is an incomplete or inadequate horizontal bracing system. Evaluate the horizontal bracing system for completeness of the system and its ability to gather all tributary forces and deliver them to the walls or frames.

7.6 Other systems. The diaphragm system does not include thin planks and/or toppings of gypsum.

The deficiency is the inadequate capacity of the diaphragm. Conforming buildings with this condition shall be placed in SPC 4.

ARTICLE 8
PROCEDURES FOR CONNECTIONS

8.0 Introduction. The connections evaluated in this article are connections between:

- Framing members and walls;
- Diaphragms and walls or frames; and
- Walls or frames and foundations.

Connections between other structural members are discussed in the appropriate article.

8.1 Connection concerns. The evaluation of these specific connections involves review of:

- Lateral support of walls that are perpendicular to the direction of the earthquake (“normal walls”);
- Transfer of shear from diaphragms to shear walls and frames that are parallel to the direction of the earthquake;
- Anchorage of walls and columns to the foundations; and
- Interconnection of elements where failure of connections would jeopardize the system.

8.2 Anchorage for normal forces.

8.2.1 Wood ledgers. The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the wall-to-diaphragm connection. Report this condition as a deficiency.

8.2.2 Wall anchorage. Exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the wall-to-diaphragm connections. Check that the anchor provides a direct, positive connection between the wall and the diaphragm for forces perpendicular to the face of the wall. Evaluate the wall anchorage, treating the wall as a portion of the building, with F_p as the demand.

8.2.3 Masonry wall anchors. Wall anchorage connections are steel anchors or straps that are developed into the diaphragm.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the wall anchors. Evaluate the wall anchorage, treating the wall as a portion of the building, with F_p as the demand.

8.2.4 Anchor spacing. The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength or the number of the anchors. Evaluate the wall anchors, treating the wall as a portion of the building, with F_p as the demand.

8.2.5 Tilt-up walls. Precast bearing walls are connected to the diaphragms for out-of-plane loads; steel anchors or straps are embedded in the walls and developed into the diaphragm.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the wall anchors. Evaluate the wall anchorage, treating the load as a portion of the building, with F_p as the demand. Check the load path between the wall anchors and the diaphragm cross tie.

8.2.6 Panel-diaphragm connections. There are at least two anchors from each precast wall panel into the diaphragm elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the number of anchors. Report this condition as a deficiency.

8.2.7 Inadequate stiffness of wall anchors. Anchors of walls to wood structural elements are installed taut and are stiff enough to prevent movement between the wall and roof.

The deficiency is in the ability of the wall anchor to prevent separations between the wall and roof sheathing that may result in out-of-plane failure of the ledger support. Inspect all anchors to see that they do not have twists, kinks, offsets, or are otherwise installed so that some movement is required before the anchor becomes effective, and that this condition may lead to cross grain bending in the ledger. Conforming buildings which fail this check shall be placed in SPC 4.

8.3 Shear transfer.

8.3.1 Transfer to shear walls. Diaphragms have sufficient capacity and are connected for transfer of loads to the shear walls.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the capacity of the connection to transfer shear. Verify the adequacy of the available diaphragm capacity.

8.3.2 Transfer to steel frames. The method used to transfer diaphragm shears to the steel frames is approved for use under lateral loads.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the capacity of the connection to transfer shear. Evaluate the capacity of the load-transfer mechanism provided, using AISC design methods or approved manufacturer's data. Compare this capacity to the assumed lateral force distribution.

8.3.3 Topping slab to walls and frames. Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled into the shear wall or frame elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the capacity of the connection to transfer shear. Evaluate the capacity of the load-transfer mechanism provided. Compare this capacity to the assumed lateral force distribution.

8.4 Vertical components to foundations.

8.4.1 Steel columns. The columns in lateral-force-resisting frames are substantially anchored to the building foundation.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the frame and the foundation. Report this condition as a deficiency.

8.4.2 Concrete columns. All longitudinal column steel is doweled into the foundation.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the column and the foundation. Report this condition as a deficiency.

8.4.3 Wood posts. There is positive connection of wood posts to the foundation and the elements being supported.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the post and the foundation. Report this condition as a deficiency.

8.4.4 Wall reinforcing. All vertical wall reinforcing is doweled into the foundation.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the wall and the foundation. Report this condition as a deficiency.

8.4.5 Shear-wall-boundary columns. The shear-wall columns are substantially anchored to the building foundation.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the shear-wall columns and the foundation. Report this condition as a deficiency.

8.4.6 Wall panels. The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the wall panel and the foundation. Report this condition as a deficiency.

8.4.7 Wood sills. All wall elements are bolted to the foundation sill at 6-foot spacing or less with proper edge and end distances for concrete and wood.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection between the wood sill and the foundation. Report this condition as a deficiency.

8.5 Interconnection of elements.

8.5.1 Girders. Girders supported by walls or pilasters have special ties to secure the anchor bolts.

The deficiency is in the strength of the pilaster at the girder anchorage. Report this condition as a deficiency. Conforming buildings that fail this check shall be placed in SPC 4.

8.5.2 Corbel bearing. If the frame girders bear on column corbels, the length of bearing is greater than 3 inches.

The deficiency is in the length of bearing. Calculate the inter-story drift. Judge the adequacy of the connections to retain their vertical load-carrying integrity at a maximum drift estimated to be equal to the drift calculated with the unreduced demand. Conforming buildings that fail this check shall be placed in SPC 4.

8.5.3 Corbel connections. The frame girders are not supported on corbels with welded elements.

The deficiency is in the strength of the connection. Check all welded connections that transfer lateral loads or are subject to frame action. Determine where connection failures would be brittle (e.g., pull-out of embedded item would occur before yield of mild steel element). Analyze structure for capacity without such connections or check such connections for seismic force amplified by factor $C_d/2$, but not less than 1.5. For connections that can allow the diaphragm to fail in a brittle manner, the R values used in computing the seismic demand shall be consistent with those for brittle systems (not to exceed $R = 2$). Conforming buildings that fail this check shall be placed in SPC 4.

8.6 Roof decking.

8.6.1 Light-gage metal, plastic or cementitious roof panels. All light-gage metal, plastic or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is the lack of connection of sufficient strength between the roof panels and the framing elements. Report this condition as a deficiency.

8.6.2 Wall panels. All wall panels (metal, fiberglass or cementitious) are properly connected to the framing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is the lack of connections of sufficient strength (to prevent a falling hazard) and flexibility (to allow for the relative displacements between the panel and the supporting frame). Report this condition as a deficiency.

ARTICLE 9
PROCEDURES FOR FOUNDATIONS AND GEOLOGIC SITE HAZARDS

9.0 Introduction. The seismic evaluation of an existing building shall include an examination of the building foundation, an assessment of the capability of the soil beneath the foundation to withstand the forces applied during an earthquake and an evaluation of any nearby geologic hazards that may affect the stability of the foundation.

9.1 Condition of foundations.

9.1.1 Foundation performance. The structure does not show evidence of excessive foundation movement such as settlement or heave that would affect its integrity or strength.

The deficiency is reduction of the integrity and strength of foundation elements by cracking, yielding, tipping or buckling of the foundation. Visually examine lower level walls, partitions, grade beams, visible footings, pile caps and the like for cracking, yielding, buckling and out-of-level conditions. Report evidence of movement as a deficiency.

9.1.2 Deterioration. There is no evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown or other reasons in a manner that would affect the integrity or strength of the structure.

The deficiency is weakening of the foundation due to deterioration, with the same consequences as discussed in Section 9.1.1. Determine if there is historical evidence in the local area of deterioration of the particular type of foundation elements in the building where site conditions are similar. Examine the visible foundation elements for evidence of loss of support as specified in Section 9.1.1.

9.2 Capacity of foundations.

9.2.1 Overturning. The ratio of the effective horizontal dimension, at the foundation level of the seismic-resisting system, to the building height (base/height) exceeds 1.4 A_v.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is the concentration of seismic inertial response into narrow elements by the seismic-resisting system, which may overcome the ability of the foundation elements, either structure or soil, to provide adequate resistance. For shallow foundations, evaluate the shear and moment capacity of the foundation elements for adequacy to resist calculated seismic forces. Evaluate the vertical bearing pressure of the soil under seismic loading conditions due to the total gravity and overturning loads and compare to two times the allowable static-bearing pressure. For deep foundations, evaluate the ultimate vertical capacity of the pile or pier under seismic loads. Compare the foundation capacity to the gravity loads plus the overturning loads.

9.2.2 Ties between foundation elements. Foundation ties adequate for seismic forces exist where footings, piles and piers are not restrained by beams, slabs, or competent soils or rock.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is the possibility of significant differential lateral deformations of the foundations. Evaluate the lateral restraint to seismic forces provided by the foundation materials or the structural ties. For shallow foundations, evaluate the horizontal capacity of the foundation soils under seismic loading conditions (the lateral resistance of the footings due to passive resistance on affected sides of the footings plus the friction on the base of the footings) and compare to the base shear of the building. In the evaluation of base friction, consideration shall be given to the effect of the vertical component of ground motion.

9.2.3 Load path at pile caps. The pile caps are capable of transferring overturning and lateral forces between the structure and individual piles in the pile cap.

The deficiency is insufficient capacity of the pile cap to transfer seismic forces from the superstructure to the individual piles. Check the moment and shear capacity to transfer uplift and lateral forces from the point of application on the pile cap to each pile. Conforming buildings which fail this check shall be placed in SPC 4.

9.2.4 Lateral force on deep foundations. Piles and piers are capable of transferring the lateral forces between the structure and the soil.

The deficiencies include inadequate flexural strength and ductility of piles or piers at the connection to the cap and the upper portion of the pile. Compare the maximum lateral resistance of soil against piles or piers and caps against the demand. For concrete piles, check for a minimal amount of longitudinal reinforcement in the upper portion of piles or piers and for hoops or ties immediately beneath the caps. Also check for confining transverse reinforcement wherever bending moments might be high, including changes in soil stiffness. Conforming buildings which fail this check shall be placed in SPC 4.

9.2.5 Pole buildings. Pole foundations have adequate embedment.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is inadequate strength of the pole foundation. Check lateral force resistance of embedded poles using conventional procedures, comparing with conventional allowable pressures times 1.5.

9.2.6 Sloping sites. The grade difference from one side of the building to another does not exceed one-half story.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. If this statement is false, include the horizontal force due to the grade difference, appropriately modified for seismic motions, with the seismic inertial force when checking sliding stability and the lateral-force-resisting system below grade.

9.3 Geologic site hazards. This section addresses geologic and local site conditions that can lead to building structural damage and threaten life safety in an earthquake. In the seismic evaluation of buildings for life-safety considerations, it will be necessary to investigate the site to establish that there are no geologic site hazards present or, if they are present, that their threat is not significant or is mitigated by the design. Requirements for engineering geologic reports are given in Section 2.1.2.

9.3.1 Liquefaction. Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building.

The deficiency is the potential for liquefaction that will result in vertical settlement and potential loss of foundation support for spread footings, or for lateral spreading of liquefied soils that can occur on nearly flat slopes and be detrimental to the foundation system. Evaluate the liquefaction potential and consequences of vertical settlement or lateral movement of the foundations. Conforming buildings which fail this check shall be placed in SPC 4.

9.3.2 Slope failure. The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating small predicted movements without failure.

Evaluate the likely movements associated with seismically induced slope failures beneath, above or adjacent to the building and their effect on the structural integrity of the building. Conforming buildings which fail this check shall be placed in SPC 4.

9.3.3 Surface fault rupture. Surface fault rupture and surface displacement at the building site are not anticipated.

Evaluate the proximity of known active faults to the building. If the potential for surface fault rupture and surface displacement at the building site is present, nonconforming buildings shall be placed in SPC 1. Conforming buildings which fail this check shall be placed in SPC 4.

ARTICLE 10
EVALUATION OF ELEMENTS THAT ARE NOT PART OF THE LATERAL-FORCE-RESISTING SYSTEM

10.0 Introduction. This article sets forth general requirements that apply to nonstructural elements related to life-safety issues. Article 11 addresses evaluation of critical nonstructural systems needed for continued hospital function following an earthquake, and assignment of buildings to Nonstructural Performance Categories.

The evaluation statements discussed in this article (and listed in the appendix) deal with life-safety concerns. Some of the statements can be answered directly. For others, further investigation will be required in accordance with evaluation procedures indicated in other articles of these regulations using seismic forces indicated in Section 2.4.6 and the appropriate C_c seismic coefficient given in Table 2.4.3.1. Also, the materials used in the nonstructural element and its connections must be considered.

10.1 Nonstructural walls. The term "nonstructural walls" refers to walls that are not part of the load-carrying system, but may become load bearing upon attachment and interaction with other elements. Evaluation must be made to determine if they are capable of resisting seismic forces required by Section 2.4.6 as well as the other requirements of these regulations.

10.1.1 Partitions.

10.1.1.1 Masonry partitions. There are no unbraced unreinforced masonry or hollow clay tile partitions in critical care areas, clinical laboratory service spaces, pharmaceutical service spaces, radiological service spaces, central and sterile supply areas, exit corridors, elevator shafts or stairwells.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check for the presence of support angles at floor and roof, and for spaces at the sides and top of the wall to provide for interaction of the structural system.

10.1.1.2 Structural separations. At structural separations, partitions in exit corridors have seismic or control joints.

Check that seismic and/or control joints have been provided at structural separations. Conforming buildings that fail this check shall be placed in SPC 4.

10.1.1.3 Partition bracing. In exit corridors, the tops of partitions that extend only to the ceiling line have lateral bracing.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Partitions extending only to ceilings may overturn or buckle due to the lack of bracing.

10.1.2 Cladding and veneer. For conforming buildings, the evaluator may consider these conditions as mitigated, and no calculations are necessary. Exterior wall panels or cladding can fall if their connections to the building frames have insufficient strength and/or ductility.

10.1.2.1 Masonry veneer. Masonry veneer is connected to the back-up with corrosion-resistant ties spaced 24 inches on center maximum with at least one tie for every $2^2/_3$ square feet.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check for the presence of the required ties.

10.1.2.2 Cladding panels in moment frame buildings. For moment frame buildings of steel or concrete, panels are isolated from the structural frame to absorb predicted interstory drift without collapse.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check the ability of the cladding panels and their connections to tolerate the story drift computed in Section 2.4.4 without an anchorage failure.

10.1.2.3 Cladding panel connections. Where bearing connections are required, there are at least two bearing connections for each cladding panel and there are at least four connections for each cladding panel capable of resisting out-of-plane forces.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Verify that an adequate number of the appropriate connection types are present for each cladding panel.

10.1.2.4 Cladding panel condition. Cladding panel connections appear to be installed properly. No connection element is severely deteriorated or corroded. There is no cracking in the panel materials indicative of substantial structural distress. There is no substantial damage to exterior cladding due to water leakage. There is no substantial damage to exterior wall cladding due to temperature movements.

Substantial deterioration can lead to loss of cladding elements or panels. Exterior walls shall be checked for deterioration. Damage due to corrosion, rotting, freezing or erosion can be concealed within the wall. Probe into the wall space, if necessary, for signs of water leakage at vulnerable interior spaces (e.g., around windows and at floor areas). Check elements that tie cladding to the backup structure and that tie the back-up structure to floor and roof slabs. Check exterior walls for cracking due to thermal movements. Check the cladding systems with appropriate reductions in member capacities. Conforming buildings that fail this check shall be placed in SPC 4.

10.1.3 Metal stud back-up systems.

10.1.3.1 General. Additional steel studs frame window and door openings. Corrosion of veneer ties, tie screws, studs and stud tracks is minimal. Stud tracks are adequately fastened to the structural frame.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Verify that adequate framing has been provided around openings in the exterior walls. Check the cladding systems with appropriate reductions in member capacities. Check the adequacy of the connection to the structural frame using the forces specified in Section 2.4.6.

10.1.3.2 Masonry veneer with stud back-up. Masonry veneer more than 30 feet above the ground is supported by shelf angles or other elements at each floor level. Masonry veneer is adequately anchored to the back-up at locations of through-wall flashing. Masonry veneer is connected to the back-up with corrosion-resistant ties spaced 24 inches on center maximum and with at least one tie for every $2^2/_3$ square feet.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check that adequate supports and ties are provided.

10.1.4 Masonry veneer with concrete block back-up.

10.1.4.1 General. The concrete block back-up qualifies as reinforced masonry.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Verify that the concrete block back-up meets the requirements of Sections 5.3.2 and 5.3.3.

10.1.4.2 Masonry veneer support. Masonry veneer more than 30 feet above the ground is supported by shelf angles or other elements at each floor level. Masonry veneer is adequately anchored to the back-up at locations of through-wall flashing. Masonry veneer is connected to the back-up with corrosion-resistant ties spaced 24 inches on center maximum and with at least one tie for every $2^2/_3$ square feet. The concrete block back-up is positively anchored to the structural frame at 4-foot maximum intervals along the floors and roofs.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check that adequate supports and ties are provided.

10.1.5 Other veneer/panel systems.

10.1.5.1 Thin stone veneer panels. Stone anchorages are adequate for computed loads.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. There are no visible cracks or weak veins in the stone. Check the adequacy of the connection to the stone anchorage using the forces specified in Section 2.4.6.

10.1.5.2 Wood/aggregate panels. There is no visible deterioration of screws or wood at panel attachment points.

The deficiency is in the strength of the connections. Determine the cause and extent of distress and check the attachment of the panels with appropriate reductions in capacity. Conforming buildings that fail this check shall be placed in SPC 4.

10.1.6 Parapets, cornices, ornamentation and appendages. There are no laterally unsupported unreinforced masonry parapets or cornices above the highest anchorage level with height/thickness ratios greater than 1.5. Concrete parapets with height/thickness ratios greater than 1.5 have vertical reinforcement. Cornices, parapets, signs and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces and other exterior wall ornamentation are reinforced and well anchored to the structural system.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. 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, becoming significant life-safety hazards. Check the adequacy of these items using the forces specified in Section 2.4.6.

10.1.7 Means of egress. Canopies are anchored and braced to prevent collapse and blockage of building exits.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check canopies for the forces specified in Section 2.4.6.

ARTICLE 11
EVALUATION OF CRITICAL NONSTRUCTURAL COMPONENTS AND SYSTEMS

11.0 Introduction. This article covers nonstructural components and systems critical to patient care.

11.01 Nonstructural evaluation procedure.

1. The nonstructural performance evaluation shall examine the respective critical nonstructural systems and elements for the planned NPC as specified in Table 11.1, "Nonstructural Performance Categories." The nonstructural evaluation process shall include the following steps:
   1. Site visit and data collection;
   2. Identification of building SPC;
   3. Identification of critical nonstructural systems for the planned NPC;
   4. Identification of critical care services housed in the building;
   5. Final evaluation for the critical nonstructural elements and systems for the planned NPC;
   6. Preparation of evaluation report; and
   7. Submittal of evaluation report to OSHPD.
2. A general acute care hospital facility may be exempted from a nonstructural evaluation upon submittal of a written statement by the hospital owner to OSHPD certifying the following conditions:
   1. The building is designated "NPC 1" in conformance with Table 11.1 "Nonstructural Performance Categories," or
   2. The building is designated "NPC 4" in conformance with Table 11.1 "Nonstructural Performance Categories" and provided:
      a) The building was designed and constructed under a building permit issued by OSHPD;
      b) All subsequent repairs, remodels, additions and alterations were performed under a permit issued by OSHPD, and
      c) Fire sprinkler systems have been retrofitted in conformance with Table 11.1, "Nonstructural Performance Categories."
3. If a hospital owner elects to obtain a higher NPC at a future date, additional nonstructural evaluations as specified in Section 11.01.1 will be required.
4. If a hospital owner sells or leases the hospital to another party, a complete nonstructural evaluation and list of all nonstructural deficiencies to achieve NPC 5 shall be submitted to the Office prior to the completion of the sale or lease.

11.1 Nonstructural performance categories. Each building shall be assigned a Nonstructural Performance Category (NPC), based upon the degree of anchorage and bracing of selected nonstructural elements and systems. This includes architectural, mechanical, electrical and hospital equipment in addition to associated conduit, ductwork, piping and machinery. NPCs are defined in Table 11.1.

11.1.1 Site visit and evaluation. The evaluator shall:

1. Visit the building to observe and record the type, nature and physical condition of the nonstructural elements and systems for the planned NPC;
2. Note the SPC of the buildings based on procedures followed in Article 2;
3. Assemble building design data including:
   a. Construction drawings, specifications and calculations, and
   b. All drawings, specifications and calculations for remodeling work.
4. During the visit, the evaluator shall:
   a. Verify existing data;
   b. Develop other needed data (e.g., measure and sketch building if necessary);
   c. Verify the critical nonstructural systems of the planned NPC;
   d. Verify the critical care areas/services; and
   e. Identify special conditions which may impact the nonstructural systems or endanger the function of the critical care areas/services.

If drawings are not available, the site visit and evaluation shall be performed as described in this section.

5. Review other data available such as assessments of building performance and function following past earthquakes;
6. Prepare a summary of data using an OSHPD approved format;
7. Perform the evaluation using the procedures in Section 11.2; and
8. Prepare a report of the findings of the evaluation using an OSHPD approved format.

11.2 Evaluation of buildings. Conforming and nonconforming buildings shall be placed in an NPC based upon the degree of anchorage and bracing for those systems and equipment specified in Table 11.1. The scope of the nonstructural evaluation may be limited to the nonstructural systems and elements specified in Table 11.1 for the planned NPC. Buildings which do not meet the requirements for NPC 2 as defined in Table 11.1 shall be placed in NPC 1.

TABLE 11.1—NONSTRUCTURAL PERFORMANCE CATEGORIES

TIMEFRAMES	NONSTRUCTURAL PERFORMANCE CATEGORY[1]	DESCRIPTION
	NPC 1	Buildings with equipment and systems not meeting the bracing and anchorage requirements of any other NPC.
January 1, 2002	NPC 2	The following systems are braced or anchored in accordance with Part 2, Title 24[1]: • communications systems, • emergency power supply, • bulk medical gas systems, • fire alarm systems and • emergency lighting equipment and signs in the means of egress.
January 1, 2008	NPC 3/NPC 3R	The building meets the criteria for NPC "2" and in critical care areas, clinical laboratory service spaces, pharmaceutical service spaces, radiological service spaces, and central and sterile supply areas, the following components meet the bracing and anchorage requirements of Part 2, Title 24[2]: • Nonstructural components, listed in the 1995 CBC, Part 2, Title 24, Table 16A-0. **Exception:** For NPC 3R, lateral bracing of suspended ceiling systems may be omitted in rooms with a floor area less than 300 square feet, provided the room is not an intensive care or coronary care unit patient room, angiography laboratory, cardiac catheterization laboratory, delivery room, operating room or post-operative recovery room. • "Equipment," as listed in the 1995 CBC, Part 2, Title 24, Table 16A-0, "Equipment," including equipment in the physical plant that service these areas. **Exceptions:** 1. Seismic restraints need not be provided for cable trays, conduit and HVAC ducting. Seismic restraints may be omitted from piping systems, provided that an approved method of preventing release of the contents of the piping system in the event of a break is provided. 2. Only elevator(s) selected to provide service to patient, surgical, obstetrical and ground floors during interruption of normal power need to meet the structural requirements of Part 2, Title 24. • Fire sprinkler systems comply with the bracing and anchorage requirements of NFPA 13, 1994 edition, or subsequent applicable standards. **Exception:** Acute care hospital facilities in both a rural area as defined by Section 70059.1, Division 5 of Title 22 and Seismic Zone 3 shall comply with the bracing and anchorage requirements of NFPA 13, 1994 edition, or subsequent applicable standards by January 1, 2013.
	NPC 4	The building meets the criteria for NPC "3" and all architectural, mechanical, electrical systems, components and equipment, and hospital equipment meet the bracing and anchorage requirements of Part 2, Title 24[2]. This category is for classification purposes of the Office of Emergency Services.
January 1, 2030	NPC 5	The building meets the criteria for NPC "4" and onsite supplies of water and holding tanks for wastewater, sufficient for 72 hours emergency operations, are integrated into the building plumbing systems. As an alternative, hook-ups to allow for the use of transportable sources of water and sanitary waste water disposal have been provided. An onsite emergency system as defined within Part 3, Title 24 is incorporated into the building electrical system for critical care areas. Additionally, the system shall provide for radiological service and an onsite fuel supply for 72 hours of acute care operation.

[1]For the purpose of NPC 2 and NPC 5, all enumerated items within Table 11.1 shall meet the requirements of Section 1632A of 2001 *California Building Code* (CBC) or equivalent provision in later version of the CBC by the specified timeframe as indicated by their respective NPC.

[2]For the purposes of NPC 3 and NPC 4, all enumerated items within Table 11.1 shall meet the requirements of the 1998 CBC, Section 1630B, by the specified timeframe. For the purposes of NPC 3R, all enumerated items within Table 11.1 shall meet the requirements of the 1995 CBC, Section 1630A, using *lp* = 1.0, by the specified timeframe.

11.2.1 Evaluation procedures for NPC 2. The following steps shall determine if the building meets the criteria for NPC 2:

a) Identify the specific nonstructural components and equipment that are subject to the requirements of NPC 2 as specified in Table 11.1;

b) Conduct an inventory of components and equipment, noting whether the items are anchored or braced;

c) Determine if the anchorage or bracing of the identified components and equipment complies with the following conditions:

1. Installed under a permit issued by OSHPD. Drawings showing the installation and bearing an OSHPD approval stamp are required to show that the installation conforms to Part 2, Title 24; or

2. Reviewed and approved by the Department of General Services, Office of Architecture and Construction, Structural Safety Section. Drawings showing: a) the installation; b) bear an Office of Architecture and Construction, Structural Safety Section approval stamp; and c) a five-digit project number on the approval that begins with the "H" prefix, are required to demonstrate that the installation conforms to Part 2, Title 24. It shall also be demonstrated by a written report submitted by the structural engineer, acceptable to the enforcement agency, that an investigation of the anchorage and bracing of components and equipment identified in Section 11.2.1(a) shows it to be constructed in reasonable conformity with these drawings.

Anchorage and bracing of elements that comply with either of these conditions are considered to meet the requirements of NPC 2.

Installation is defined as that which shows the size and type of material for all components of the system, including the anchor or fastener manufacturer (if proprietary), type, total number and embedment if connected to structural concrete, masonry or wood.

d) If the components and equipment inventoried in 11.2.1(b) is anchored or braced, but does not meet the requirements of Section 11.2.1(c), determine if the bracing and anchorage is sufficient to meet the code requirements specified in Table 11.1. The bracing capacity shall be determined by calculations based upon information shown in the construction documents. If these documents are incomplete or unavailable, the evaluation shall be based on the as-built conditions, with the capacity of fasteners to masonry, concrete or wood determined by approved tests; and

e) If any of the items inventoried in 11.2.1(b) are unanchored or inadequately braced as determined by Section 11.2.1(d), the building shall be placed in NPC 1.

11.2.2 Evaluation procedures for NPC 3 and NPC 3R. The following steps shall determine if the building meets the criteria for NPC 3 or NPC 3R:

a) Identify the specific nonstructural components and equipment that are subject to the requirements of NPC 2 and NPC 3 or NPC 3R;

b) Conduct an inventory of components and equipment specified in Table 11.1, NPC 3 and NPC 3 R, noting whether the components and equipment are anchored or braced;

Exception: Any general acute care hospital facility located in both a "rural area" as defined in Section 70059.1, Division 5, Title 22 and Seismic Zone 3 pursuant to 1995 *California Building Code* (CBC) or later version of the CBC shall comply with the fire sprinkler system anchorage and bracing requirements of NFPA 13, 1994 edition or subsequent standard by January 1, 2013.

c) Determine the level of NPC 3 conformance desired.

1. Buildings classified as SPC 1 or SPC 2 are permitted to meet the NPC 3 performance level, or the NPC 3R performance level. See also Section 11.2.3(c).

2. Buildings classified as SPC 3 or higher must meet the NPC 3 performance level.

d) Determine if the anchorage or bracing of the identified components and equipment complies with the following conditions:

1. Installed under a permit issued by OSHPD. Drawings showing the installation and bearing an OSHPD approval stamp are required to show that the installation conforms to Part 2, Title 24; or

2. Reviewed and approved by the Department of General Services, Office of Architecture and Construction, Structural Safety Section. Drawings showing: a) the installation; b) bear an Office of Architecture and Construction, Structural Safety Section approval stamp; and c) a five-digit project number on the approval stamp that begins with an "H" prefix, are required to demonstrate that the installation conforms to Part 2, Title 24. It shall also be demonstrated by a written report submitted by the structural engineer, acceptable to the enforcement agency, that an investigation of the anchorage and bracing of components and equipment identified in Section 11.2.2(a) shows it to be constructed in reasonable conformity with these drawings.

Anchorage and bracing of elements that comply with either of these conditions are considered to meet the requirements of NPC 2 and NPC 3 or NPC 3R.

Installation is defined as that which shows the size and type of material for all components of the system including the anchor or fastener manufacturer (if proprietary), type, total number and embedment if connected to structural concrete, masonry or wood.

e) If the components and equipment inventoried in 11.2.2(b) are anchored or braced, but do not meet the requirements of Section 11.2.2(d), determine if the bracing and anchorage is sufficient to meet the code requirements specified in Table 11.1 for NPC 3 or NPC 3R. The bracing capacity shall be determined by calculations based upon information shown in the construction documents. If these documents are incomplete or unavailable, the evaluation shall be based on the as-built conditions, with the capacity of fasteners to masonry, concrete, or wood determined by approved tests. For NPC 3R, the investigation of the adequacy of anchorage and bracing may be limited to the connection of the component or equipment to the support when the total reaction at the point of support (including the application of F_p) is less than:

1. 250 pounds for components or equipment attached to light frame walls. For the purposes of this requirement, the sum of the absolute value of all reactions due to component loads on a single stud shall not exceed 250 pounds.
2. 1,000 pounds for components or equipment attached to roofs, or walls of reinforced concrete or masonry construction.
3. 2,000 pounds for components or equipment attached to floors or slabs-on-grade.

Exception: If the anchorage or bracing is configured in a manner that results in significant torsion on a supporting structural element, the effects of the nonstructural reaction force on the structural element shall be considered in the anchorage design.

f) If any of the items inventoried in 11.2.2(b) are inadequately anchored or braced, as determined by Section 11.2.2(d), the building shall be placed in NPC 2.

11.2.3 Evaluation procedures for NPC 4. The following steps shall be followed to determine if the building meets the criteria for NPC 4:

a) Identify the specific nonstructural components and equipment that are subject to the requirements of NPC 2 through NPC 4;

b) Conduct an inventory of components and equipment specified in Table 11.1, NPC 2 through NPC 4, noting whether the components and equipment are anchored or braced;

c) Determine if the anchorage or bracing of the identified components and equipment complies with one of the following conditions:

1. Installed under a permit issued by OSHPD. Drawings showing the installation and bearing an OSHPD approval stamp are required to show that the installation conforms to Part 2, Title 24. Installation or retrofit of components that were designed to meet NPC 3R requirements must be shown to meet the anchorage and bracing requirements of the *California Building Code* for new construction. Components designed to meet NPC 3R requirements that do not meet the anchorage and bracing requirements for new construction shall be retrofitted to meet those requirements; or
2. Reviewed and approved by the Department of General Services, Office of Architecture and Construction, Structural Safety Section. Drawings showing: a) the installation; b) bear an Office of Architecture and Construction, Structural Safety Section approval stamp; and c) a five-digit project number on the approval stamp that begins with an "H" prefix, are required to demonstrate that the installation conforms to Part 2, Title 24. It shall also be demonstrated by a written report submitted by the structural engineer, acceptable to the enforcement agency, that an investigation of the anchorage and bracing of components and equipment identified in Section 11.2.3(a) shows it to be constructed in reasonable conformity with these drawings.

Anchorage and bracing of elements that comply with either of these conditions are considered to meet the requirements of NPC 4.

Installation is defined as that which shows the size and type of material for all components of the system including the anchor or fastener manufacturer (if proprietary), type, total number and embedment if connected to structural concrete, masonry or wood.

d) If the components and equipment inventoried in 11.2.3(b) are anchored or braced, but do not meet the requirements of Section 11.2.3(c), determine if the bracing and anchorage is sufficient to meet the code requirements specified in Table 11.1. The bracing capacity shall be determined by calculations based upon information shown in the construction documents. If these documents are incomplete or unavailable, the evaluation shall be based on the as-built conditions, with the capacity of fasteners to masonry, concrete or wood determined by approved tests; and

e) If any of the items inventoried in 11.2.3(b)is unanchored or inadequately braced as determined by Section 11.2.3(d), the building shall be placed in NPC 3.

11.2.4 Evaluation procedures for NPC 5. The following steps shall determine if the building meets the criteria for NPC 5:

a) Identify the specific nonstructural components and equipment that are subject to the requirements of NPC 2 through NPC 5;

b) Conduct an inventory of components and equipment specified in Table 11.1, NPC 2 through NPC 5, noting whether the components and equipment are anchored or braced;

c) Determine if the anchorage or bracing of the identified components and equipment complies with the following conditions:

1. Installed under a permit issued by OSHPD. Drawings showing the installation and bearing an OSHPD approval stamp are required to show that the installation conforms to Part 2, Title 24; or
2. Reviewed and approved by the Department of General Services, Office of Architecture and Construction, Structural Safety Section. Drawings showing: a) the installation; b) bear an Office of Architecture and Construction, Structural Safety Section approval stamp; and c) a five-digit project number on the approval stamp that begins with an "H" prefix, are required to demonstrate that the installation conforms to Part 2, Title 24. It shall also be demon-

strated by a written report submitted by the structural engineer, acceptable to the enforcement agency, that an investigation of the anchorage and bracing of components and equipment identified in Section 11.2.4(a) shows it to be constructed in reasonable conformity with these drawings.

Anchorage and bracing of elements that comply with either of these conditions are considered to meet the requirements of NPC 5.

Installation is defined as that which shows the size and type of material for all components of the system including the anchor or fastener manufacturer (if proprietary), type, total number and embedment if connected to structural concrete, masonry or wood.

d) If the components and equipment inventoried in 11.2.4(b) are anchored or braced, but do not meet the requirements of Section 11.2.4(c), determine if the bracing and anchorage is sufficient to meet the code requirements specified in Table 11.1. The bracing capacity shall be determined by calculations based upon information shown in the construction documents. If these documents are incomplete or unavailable, the evaluation shall be based on the as-built conditions, with the capacity of fasteners to masonry, concrete or wood determined by approved tests; and

e) If any of the items inventoried in 11.2.4(b) is inadequately anchored or braced as determined by 11.2.4(d), the building shall be placed in NPC 4.

11.3 Testing requirements for evaluating the performance of existing mechanical fasteners. A testing program shall be instituted to determine the capacity of mechanical fasteners used to anchor nonstructural components including the bracing of pipes, ducts and conduit, and the attachment of equipment and other components listed in the 1995 CBC, Part 2, Title 24, Table 16A-0. Anchors shall be categorized as either seismic bracing of pipes ducts or conduit or equipment and other component anchors.

11.3.1 Anchors used in the seismic bracing of pipes, ducts or conduit. For anchors used in the seismic bracing of pipes, ducts or conduit, the following shall apply:

1. Twenty percent of the anchors (20 minimum) of a given size and type (wedge, shell and sleeve for expansion bolts), at each level of the structure shall be tension tested to three times the maximum calculated design load specified in Section 1630B of 1998 *California Building Code* (CBC) or equivalent provision in later version of the CBC but not less than 500 pounds. A minimum of one anchor in any 4-bolt group shall be tested assuming an equal distribution of the calculated force to the bolt group. One-quarter ($^1/_4$)-inch diameter anchors need not be tested. Where none of the anchors in the group have calculated tension, testing shall consist of torque testing.

   Exception: Internally threaded anchors, such as shell-type anchors, shall be tested to four times the maximum calculated design loads. Attachment hardware shall be shimmed or removed prior to testing so that it does not prevent the possible withdrawal of the anchor.

2. If an anchor fails the tension test, 20 anchors, installed by the same trade, in the immediate vicinity of the failed anchor shall be tested prior to resuming to a 20 percent sampling rate for testing.

11.3.2 Anchors used in the attachment of equipment and other components. For anchors used in the attachment of equipment and other components listed in the 1995 CBC, Part 2, Title 24, Table 16A-0, the following shall apply:

1. A minimum of one anchor of a given size shall be tension tested for each piece of equipment or other component under consideration. Where the number of anchors for the piece of equipment or component exceeds four, a minimum of 20 percent of the anchors shall be tension tested. Where none of the anchors in the group have calculated tension, testing shall consist of torque testing.
2. The tension test load shall be three times the maximum tension force calculated for an anchor in the attachment group using the design loads specified in Section 1630B of 1998 *California Building Code* (CBC) or equivalent provision in later version of the CBC or 500 pounds minimum. One-quarter ($^1/_4$)-inch diameter anchors need not be tested.

   Exception: Internally threaded anchors, such as shell type anchors, shall be tested to four times the maximum calculated design loads. Attachment hardware shall be shimmed or removed prior to testing so that it does not prevent the possible withdrawal of the anchor.

3. If a single anchor fails, all anchors in the attachment group shall be tested. If two or more anchors fail, the component shall be retrofitted for the forces as for new construction.

11.3.3 Tension testing procedure.

1. Testing of anchors shall be accomplished by the application of externally applied direct tension force to the anchor. The testing apparatus shall not restrict the probable shear cone failure surface of the concrete or masonry.
2. Torque testing is not permitted in lieu of tension testing unless specifically allowed in these provisions.
3. A failure is defined when the tension load on the anchor produces a slip of $^1/_8$ inch, a shear cone failure in the concrete or masonry, concrete splitting, or fracture of the steel anchor itself prior to attaining the test load value.

   Exception: For internally threaded anchors, the allowable slip shall not exceed $^1/_{16}$ inch.

11.3.4 Alternate test criteria. In lieu of testing in accordance with Section 11.3.1 or 11.3.2, a test load may be established by the evaluating engineer. The allowable load that the anchor can resist shall be determined by dividing the test load by the appropriate factors noted in Section 11.3.1 or 11.3.2. No one-third increase is permitted for seismic or wind loads.

11.3.5 Allowable shear loads. Allowable shear loads on anchors shall be determined by either of the following:

1. Shear values listed in Table 19B-E of 1998 *California Building Code* (CBC) or equivalent provision in later version of the CBC, or

2. Shear values shall be obtained by analysis using Strength Design of Anchorage to Concrete, Section A.6, published by the Portland Cement Association, 1999, with the specified reduction coefficient(s) to convert the “strength” values to allowable stress design values of 1.7.

APPENDIX
GENERAL SETS OF EVALUATION STATEMENTS

EVALUATION STATEMENTS FOR THE BASIC BUILDING SYSTEM

Address the following evaluation statements, marking each either true (T), false (F) or not applicable (N/A). Statements that are found to be true identify issues that are acceptable according to the criteria of these regulations; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the section number indicated in parentheses at the end of the statement.

Building system

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Section 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Section 3.2)

Configuration

T F N/A WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Section 3.3.1)

T F N/A SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Section 3.3.2)

T F N/A GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Section 3.3.3)

T F N/A MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Section 3.3.4)

T F N/A VERTICAL DISCONTINUITIES: All shear walls, infilled walls and frames are continuous to the foundation. (Section 3.3.5)

T F TORSION: 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 dimension. (Section 3.3.6)

Adjacent buildings

T F ADJACENT BUILDINGS: There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated. (Section 3.4)

Deflection incompatibility

T F DEFLECTION INCOMPATIBILITY: Column and beam assemblies that are not part of the lateral-force-resisting system (i.e., gravity load-resisting frames) are capable of accommodating imposed building drifts, including amplified drift caused by diaphragm deflections, without loss of vertical load-carrying capacity. (Section 3.5)

Short "captive" columns

T F SHORT "CAPTIVE" COLUMNS: There are no columns with height-to-depth ratios less than 75 percent of the nominal height-to-depth ratios of the typical columns at that level. (Section 3.6)

Materials and conditions

T F N/A DETERIORATION OF WOOD: None of the wood members shows signs of decay, shrinkage, splitting, fire damage or sagging, and none of the metal accessories is deteriorated, broken or loose. (Section 3.7.1)

T F N/A OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (Section 3.7.2)

T F N/A DETERIORATION OF STEEL: There is no significant visible rusting, corrosion or other deterioration in any of the steel elements in the vertical- or lateral-force-resisting system. (Section 3.7.3)

T F N/A DETERIORATION OF CONCRETE: There is no visible deterioration of concrete or reinforcing steel in any of the frame elements. (Section 3.7.4)

T F N/A POST-TENSIONING ANCHORS: There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors have not been used. (Section 3.7.5)

T F N/A CONCRETE WALL CRACKS: All diagonal cracks in the wall elements are 1.0 mm or less in width, are in isolated locations, and do not form an X pattern. (Section 3.7.6)

T F N/A CRACKS IN BOUNDARY COLUMNS: There are no diagonal cracks wider than 1.0 mm in concrete columns that encase the masonry infills. (Section 3.7.7)

Materials and conditions—cont.

T F N/A PRECAST CONCRETE WALLS: There is no significant visible deterioration of concrete or reinforcing steel or evidence of distress, especially at the connections. (Section 3.7.8)

T F N/A MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Section 3.7.9)

T F N/A MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Section 3.7.10)

T F N/A CRACKS IN INFILL WALLS: There are no diagonal cracks in the infilled walls that extend throughout a panel or are greater than 1.0 mm wide. (Section 3.7.11)

EVALUATION STATEMENTS FOR VERTICAL SYSTEMS RESISTING LATERAL FORCES

Address the following evaluation statements, marking each either true (T), false (F) or not applicable (N/A). Statements that are found to be true identify issues that are acceptable according to the criteria of these regulations; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the section number indicated in parentheses at the end of the statement.

MOMENT FRAMES

Frames with infill walls

T F N/A INTERFERING WALLS: All infill walls placed in the moment frames are isolated from the structural elements. (Section 4.1.1)

Steel moment frames

T F N/A DRIFT CHECK: The building satisfies the Quick Check of the frame drift. (Section 4.2.1)

T F N/A COMPACT MEMBERS: All moment frame elements meet the compact section requirements of the basic AISC documents. (Section 4.2.2)

T F N/A BEAM PENETRATIONS: All openings in frame-beam webs have a depth less than one fourth of the beam depth and are located in the center half of the frame beams. (Section 4.2.3)

T F N/A MOMENT CONNECTIONS: All beam-column connections in the lateral-force-resisting moment frame have full-penetration flange welds and a bolted or welded web connection. (Section 4.2.4)

T F N/A COLUMN SPLICES: All column splice details of the moment-resisting frames include connection of both flanges and the web. (Section 4.2.5)

T F N/A JOINT WEBS: All web thicknesses within joints of moment-resisting frames meet the AISC criteria for web shear. (Section 4.2.6)

T F N/A GIRDER FLANGE CONTINUITY PLATES: There are girder flange continuity plates at joints. (Section 4.2.7)

T F N/A STRONG COLUMN/WEAK BEAM: At least one half of the joints are strong column/weak beam (33 percent on every line of moment frame). Roof joints need not be considered. (Section 4.2.8)

T F N/A OUT-OF-PLANE BRACING: Beam-column joints are braced out-of-plane. (Section 4.2.9)

T F N/A PRE-NORTHRIDGE EARTHQUAKE WELDED MOMENT FRAME JOINTS: Welded steel moment frame beam-column joints are designed and constructed in accordance with recommendations in FEMA 267, Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures, August 1995. (Section 4.2.10)

Concrete moment frames

T F N/A SHEARING STRESS CHECK: The building satisfies the Quick Check of the average shearing stress in the columns. (Section 4.3.1)

T F N/A DRIFT CHECK: The building satisfies the Quick Check of story drift. (Section 4.3.2)

T F N/A PRESTRESSED FRAME ELEMENTS: The lateral-load-resisting frames do not include any pre-stressed or post-tensioned elements. (Section 4.3.3)

T F N/A JOINT ECCENTRICITY: There are no eccentricities larger than 20 percent of the smallest column plan dimension between girder and column center-lines. (Section 4.3.4)

T F N/A NO SHEAR FAILURES: The shear capacity of frame members is greater than the moment capacity. (Section 4.3.5)

T F N/A STRONG COLUMN/WEAK BEAM: The moment capacity of the columns appears to be greater than that of the beams. (Section 4.3.6)

T F N/A STIRRUP AND TIE HOOKS: The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more. (Section 4.3.7)

T F N/A COLUMN-TIE SPACING: Frame columns have ties spaced at $d/4$ or less throughout their length and at $8d_b$, or less at all potential plastic hinge regions. (Section 4.3.8)

T F N/A COLUMN-BAR SPLICES: All column bar lap splice lengths are greater than $35d_b$, long and are enclosed by ties spaced at $8d_b$, or less. (Section 4.3.9)

T F N/A BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25 percent of the steel provided at the joints for either positive or negative moment is continuous throughout the members. (Section 4.3.10)

Concrete moment frames—cont.

T F N/A BEAM-BAR SPLICES: The lap splices for the longitudinal beam reinforcing are located within the center half of the member lengths or in the vicinity of potential plastic hinges. (Section 4.3.11)

T F N/A STIRRUP SPACING: All beams have stirrups spaced at $d/2$ or less throughout their length and at $8d_b$ or less at potential hinge locations. (Section 4.3.12)

T F N/A BEAM TRUSS BARS: Bent-up longitudinal steel is not used for shear reinforcement. (Section 4.3.13)

T F N/A JOINT REINFORCING: Column ties extend at their typical spacing through all beam-column joints at exterior columns. (Section 4.3.14)

T F N/A FLAT SLAB FRAMES: The system is not a frame consisting of a flat slab/plate without beams. (Section 4.3.15)

Precast concrete moment frames

T F N/A PRECAST FRAMES: The lateral loads are not resisted by precast concrete frame elements. (Section 4.4.1)

T F N/A PRECAST CONNECTIONS: For buildings with concrete shear walls, the connection between precast frame elements such as chords, ties and collectors in the lateral-force-resisting system can develop the capacity of the connected members. (Section 4.4.2)

Frames not part of the lateral-force-resisting system

T F N/A COMPLETE FRAMES: The steel or concrete frames form a complete vertical load-carrying system. (Section 4.5.1)

SHEAR WALLS

Concrete shear walls

T F N/A SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the shear walls. (Section 5.1.1)

T F N/A OVERTURNING: All shear walls have h_w/l_w ratios less than 4 to 1. (Section 5.1.2)

T F N/A COUPLING BEAMS: The stirrups in all coupling beams are spaced at $d/2$ or less and are anchored into the core with hooks of 135 degrees or more. (Section 5.1.3)

T F N/A COLUMN SPLICES: Steel column splice details in shear wall boundary elements can develop the tensile strength of the column. (Section 5.1.4)

T F N/A WALL CONNECTIONS: There is positive connection between the shear walls and the steel beams and columns. (Section 5.1.5)

T F N/A CONFINEMENT REINFORCING: For shear walls with *hw/lw* greater than 2.0, the boundary elements are confined with spirals or ties with spacing less then $8d_b$. (Section 5.1.6)

T F N/A REINFORCING STEEL: The area of reinforcing steel for concrete walls is greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes and the maximum spacing of reinforcing steel is 18 inches. (Section 5.1.7)

T F N/A REINFORCING AT OPENINGS: There is special wall reinforcement around all openings. (Section 5.1.8)

Precast concrete shear walls

T F N/A PANEL-TO-PANEL CONNECTIONS: Adjacent wall panels are not connected by welded steel inserts. (Section 5.2.1)

T F N/A WALL OPENINGS: Openings constitute less than 75 percent of the length of any perimeter wall with the wall piers having h_w/l_w ratios of less than 2.0. (Section 5.2.2)

T F N/A COLLECTORS: Wall elements with openings larger than a typical panel at a building corner are connected to the remainder of the wall with collector reinforcing. (Section 5.2.3)

Reinforced masonry shear walls

T F N/A SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Section 5.3.1)

T F N/A REINFORCING: The total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions, the spacing of reinforcing steel is less than 48 inches and all vertical bars extend to the top of the walls. (Section 5.3.2)

T F N/A REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Section 5.3.3)

Unreinforced masonry shear walls

T F N/A SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (Section 5.4.1)

T F N/A MASONRY LAY-UP: Filled collar joints of multi-wythe masonry walls have negligible voids. (Section 5.4.2)

Infill walls in frames

T F N/A PROPORTIONS: The height/thickness ratio of the wall panels is as follows (Section 5.5.1):

One-story building $h_w/t < 14$

Multistory building

Top story $h_w/t < 9$

Other stories $h_w/t < 20$

T F N/A SOLID WALLS: The infill walls are not of cavity construction. (Section 5.5.2)

T F N/A CONTINUOUS WALLS: The infill walls are continuous to the soffits of the frame beams. (Section 5.5.3)

T F N/A WALL CONNECTIONS: All infill panels are constructed to encompass the frames around their entire perimeter. (Section 5.5.4)

Walls in wood-frame buildings

T F N/A SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the wood shear walls. (Section 5.6.1)

T F N/A OPENINGS: Walls with garage doors or other large openings are braced with plywood shear walls or are supported by adjacent construction through substantial positive ties. (Section 5.6.2)

T F N/A WALL REQUIREMENTS: All walls supporting tributary area of 24 to 100 square feet per foot of wall are plywood sheathed with proper nailing, or rod braced and have a height-to-depth (H/D) ratio of 1 to 1 or less, or have properly detailed and constructed hold downs. (Section 5.6.3)

T F N/A CRIPPLE WALLS: All exterior cripple walls below the first floor level are braced to the foundation with shear elements. (Section 5.6.4)

T F N/A NARROW SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2 to 1 do not resist forces developed in the building. (Section 5.6.5)

T F N/A STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multistory buildings do not rely on exterior stucco walls as the primary lateral-force-resisting system. (Section 5.6.6)

T F N/A PLASTER OR GYPSUM WALLBOARD SHEAR WALLS: Interior plaster or gypsum wallboard is not being used for shear walls in buildings over one story in height. (Section 5.6.7)

BRACED FRAMES

Concentric braced frames

T F N/A STRESS CHECK: The building satisfies the Quick Check of the stress in the diagonals. (Section 6.1.1)

T F N/A STIFFNESS OF DIAGONALS: All diagonal elements required to carry compression have *Kl/r* ratios less than 120. (Section 6.1.2)

T F N/A TENSION-ONLY BRACES: Tension-only braces are not used as the primary diagonal bracing elements in structures over two stories in height. (Section 6.1.3)

T F N/A CHEVRON BRACING: The bracing system does not include chevron-, V- or K-braced bays. (Section 6.1.4)

T F N/A CONCENTRIC JOINTS: All the diagonal braces frame into the beam-column joints concentrically. (Section 6.1.5)

T F N/A CONNECTION STRENGTH: All the brace connections are able to develop the yield capacity of the diagonals. (Section 6.1.6)

T F N/A COLUMN SPLICES: All column splice details of the braced frames can develop the column yield capacity. (Section 6.1.7)

T F N/A CONCRETE BRACED FRAMES: None of the braces in the framing system are of reinforced concrete construction. (Section 6.1.8)

Eccentric braced frames

T F N/A LINK BEAM LOCATION: The link beams are not connected to the columns. (Section 6.2.1)

EVALUATION STATEMENTS FOR DIAPHRAGMS

Address the following evaluation statements, marking each either true (T), false (F) or not applicable (N/A). Statements that are found to be true identify issues that are acceptable according to the criteria of these regulations; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the section number indicated in parentheses at the end of the statement.

General

T F N/A PLAN IRREGULARITIES: There is significant tensile capacity at reentrant corners or other locations of plan irregularities. (Section 7.1.1)

T F N/A CROSS TIES: There are continuous cross ties between diaphragm chords. (Section 7.1.2)

T F N/A REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. (Section 7.1.3)

T F N/A OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than 25 percent of the wall length, and the available length appears sufficient. (Section 7.1.4)

General—cont.

T F N/A OPENINGS AT BRACED FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25 percent of the length of the bracing. (Section 7.1.5)

T F N/A OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than 8 feet long. (Section 7.1.6)

Wood diaphragms

T F N/A SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than 2 to 1. (Section 7.2.1)

T F N/A SPANS: All diaphragms with spans greater than 24 feet have plywood or diagonal sheathing. Structures in Building Type 2 may have rod-braced systems. (Section 7.2.2)

T F N/A UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans of less than 40 feet and have span/depth ratios less than or equal to 3 to 1. (Section 7.2.3)

T F N/A SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Section 7.2.4)

T F N/A DIAPHRAGM CONTINUITY: None of the diaphragms are composed of split-level floors or, in wood commercial or industrial buildings, have expansion joints. (Section 7.2.5)

T F N/A CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Section 7.2.6)

Metal deck diaphragms

T F N/A DECK TOPPING: All metal deck roofs have a reinforced concrete topping slab. (Section 7.3.1)

T F N/A UNTOPPED DIAPHRAGMS: Untapped metal deck diaphragms consist of horizontal spans of less than 40 feet and have span/depth ratios less than or equal to 3 to 1. (Section 7.3.2)

Precast concrete diaphragms

T F N/A TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (Section 7.4.1)

T F N/A CONTINUITY OF TOPPING SLAB: The topping slab continues uninterrupted through the interior walls and into the exterior walls or is provided with dowels with a total area equal to the topping slab reinforcing. (Section 7.4.2)

Horizontal bracing

T F N/A HORIZONTAL BRACING: Horizontal bracing forms a complete system of adequate capacity. (Section 7.5.1)

Other systems

T F N/A OTHER SYSTEMS: The diaphragm system does not include thin planks and/or toppings of gypsum. (Section 7.6.1)

EVALUATION STATEMENTS FOR STRUCTURAL CONNECTIONS

Address the following evaluation statements, marking each either true (T), false (F) or not applicable (N/A). Statements that are found to be true identify issues that are acceptable according to the criteria of these regulations; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the section number indicated in parentheses at the end of the statement.

Anchorage for normal forces

T F N/A WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Section 8.2.1)

T F N/A WALL ANCHORAGE: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads. (Section 8.2.2)

T F N/A MASONRY WALL ANCHORS: Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (Section 8.2.3)

T F N/A ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Section 8.2.4)

T F N/A TILT-UP WALLS: Precast-bearing walls are connected to the diaphragms for out-of-plane loads; steel anchors or straps are embedded in the walls and developed into the diaphragm. (Section 8.2.5)

T F N/A PANEL-DIAPHRAGM CONNECTION: There are at least two anchors from each precast wall panel into the diaphragm elements. (Section 8.2.6)

T F N/A INADEQUATE STIFFNESS OF WALL ANCHORS: Anchors of walls to wood structural elements are installed taut and are stiff enough to prevent movement between the wall and roof. (Section 8.2.7)

Shear transfer

T F N/A TRANSFER TO SHEAR WALLS: Diaphragms are reinforced and connected for transfer of loads to the shear walls. (Section 8.3.1)

T F N/A TRANSFER TO STEEL FRAMES: The method used to transfer diaphragm shears to the steel frames is approved for use under lateral loads. (Section 8.3.2)

T F N/A TOPPING SLAB TO WALLS AND FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled into the shear wall or frame elements. (Section 8.3.3)

Vertical components

T F N/A STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation. (Section 8.4.1)

T F N/A CONCRETE COLUMNS: All longitudinal column steel is doweled into the foundation. (Section 8.4.2)

T F N/A WOOD POSTS: There is positive connection of wood posts to the foundation and the elements being supported. (Section 8.4.3)

T F N/A WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Section 8.4.4)

T F N/A SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (Section 8.4.5)

T F N/A WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (Section 8.4.6)

T F N/A WOOD SILLS: All wall elements are bolted to the foundation sill at 6-foot spacing or less with proper edge distance for concrete and wood. (Section 8.4.7)

Interconnection of elements

T F N/A GIRDERS: Girders are supported by walls, or pilasters have special ties to secure the anchor bolts. (Section 8.5.1)

T F N/A CORBEL BEARING: If the frame girders bear on column corbels, the length of bearing is greater than 3 inches. (Section 8.5.2)

T F N/A CORBEL CONNECTIONS: The frame girders are not supported on corbels with welded elements. (Section 8.5.3)

Roof decking

T F N/A LIGHT-GAGE METAL, PLASTIC OR CEMENTITIOUS ROOF PANELS: All light-gage metal, plastic or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center. (Section 8.6.1)

T F N/A WALL PANELS: All wall panels (metal, fiberglass or cementitious) are properly connected to the wall framing. (Section 8.6.2)

EVALUATION STATEMENTS FOR FOUNDATIONS AND GEOLOGIC SITE HAZARDS

Address the following evaluation statements, marking each either true (T), false (F) or not applicable (N/A). Statements that are found to be true identify issues that are acceptable according to the criteria of these regulations; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the section number indicated in parentheses at the end of the statement.

Condition of foundations

T F FOUNDATION PERFORMANCE: The structure does not show evidence of excessive foundation movement such as settlement or heave that would affect its integrity or strength. (Section 9.1.1)

T F DETERIORATION: There is no evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown or other reasons in a manner that would affect the integrity or strength of the structure. (Section 9.1.2)

Capacity of foundations

T F OVERTURNING: The ratio of the effective horizontal dimension, at the foundation level of the seismic-resisting system to the building height (base/height) exceeds 1.4AV. (Section 9.2.1)

T F TIES BETWEEN FOUNDATION ELEMENTS: Foundation ties adequate for seismic forces exist where footings, piles and piers are not restrained by beams, slabs or competent soils or rock. (Section 9.2.2)

T F N/A LOAD PATH AT PILE CAPS: The pile caps are capable of transferring overturning and lateral forces between the structure and individual piles in the pile cap. (Section 9.2.3)

T F N/A LATERAL FORCE ON DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. (Section 9.2.4)

T F N/A POLE BUILDINGS: Pole foundations have adequate embedment. (Section 9.2.5)

T F SLOPING SITES: The grade difference from one side of the building to another does not exceed one-half story. (Section 9.2.6)

Geologic site hazards

T F N/A LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building. (Section 9.3.1)

T F SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating small, predicted movements without failure. (Section 9.3.2)

T F SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Section 9.3.3)

EVALUATION STATEMENTS FOR ELEMENTS THAT ARE NOT PART OF THE LATERAL-FORCE-RESISTING SYSTEM

Address the following evaluation statements, marking each either true (T), false (F) or not applicable (N/A). Statements that are found to be true identify issues that are acceptable according to the criteria of these regulations; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the section number indicated in parentheses at the end of the statement.

NONSTRUCTURAL WALLS

Partitions

T F N/A MASONRY PARTITIONS: There are no unbraced unreinforced masonry or hollow clay tile partitions in critical care areas, clinical laboratory service spaces, pharmaceutical service spaces, radiological service spaces, and central and sterile supply areas, exit corridors, elevator shafts or stairwells. (Section 10.1.1.1)

T F N/A STRUCTURAL SEPARATIONS: At structural separations, partitions in exit corridors have seismic or control joints. (Section 10.1.1.2)

T F N/A PARTITION BRACING: In exit corridors, the tops of partitions that extend only to the ceiling line have lateral bracing. (Section 10.1.1.3)

Cladding and veneer

T F N/A MASONRY VENEER: Masonry veneer is connected to the back-up with corrosion-resistant ties spaced 24 inches on center maximum with at least one tie for every $2^2/_3$ square feet. (Section 10.1.2.1)

T F N/A CLADDING PANELS IN MOMENT FRAME BUILDINGS: For moment frame buildings of steel or concrete, panels are isolated from the structural frame to absorb predicted interstory drift without collapse. (Section 10.1.2.2)

T F N/A CLADDING PANEL CONNECTIONS: Where bearing connections are required, there are at least two bearing connections for each cladding panel, and there are at least four connections for each cladding panel capable of resisting out-of-plane forces. (Section 10.1.2.3)

T F N/A CLADDING PANEL CONDITION: Cladding panel connections appear to be installed properly. No connection element is severely deteriorated or corroded. There is no cracking in the panel materials indicative of substantial structural distress. There is no substantial damage to exterior cladding due to water leakage. There is no substantial damage to exterior wall cladding due to temperature movements. (Section 10.1.2.4)

Metal stud back-up systems

T F N/A GENERAL: Additional steel studs frame window and door openings. Corrosion of veneer ties, tie screws, studs and stud tracks is minimal. Stud tracks are adequately fastened to the structural frame. (Section 10.1.3.1)

T F N/A MASONRY VENEER WITH STUD BACK-UP: Masonry veneer more than 30 feet above the ground is supported by shelf angles or other elements at each floor level. Masonry veneer is adequately anchored to the back-up at locations of through-wall flashing. Masonry veneer is connected to the backup with corrosion-resistant ties spaced 24 inches on center maximum and with at least one tie for every $2^2/_3$ square feet. (Section 10.1.3.2)

T F N/A MASONRY VENEER WITH CONCRETE BLOCK BACK-UP—GENERAL: The concrete block back-up qualifies as reinforced masonry. (Section 10.1.4.1)

T F N/A MASONRY VENEER SUPPORT: Masonry veneer more than 30 feet above the ground is supported by shelf angles or other elements at each floor level. Masonry veneer is adequately anchored to the back-up at locations of through-wall flashing. Masonry veneer is connected to the back-up with corrosion-resistant ties spaced 24 inches on center maximum and with at least one tie for every 22/3 square feet. The concrete block back-up is positively anchored to the structural frame at 4-foot maximum intervals along the floors and roofs. (Section 10.1.4.2)

Other veneer/panel systems

T F N/A THIN STONE VENEER PANELS: Stone anchorages are adequate for computed loads. (Section 10.1.5.1)

T F N/A WOOD/AGGREGATE PANELS: There is no visible deterioration of screws or wood at panel attachment points. (Section 10.1.5.2)

Parapets, cornices, ornamentation and appendages

T F N/A PARAPETS, CORNICES, ORNAMENTATION AND APPENDAGES: There are no laterally unsupported unreinforced masonry parapets or cornices above the highest anchorage level with height/thickness ratios greater than 1.5. Concrete parapets with height/thickness ratios greater than 1.5 have vertical reinforcement. Cornices, parapets, signs and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces and other exterior wall ornamentation are reinforced and well anchored to the structural system. (Section 10.1.6)

T F N/A MEANS OF EGRESS: Canopies are anchored and braced to prevent collapse and blockage of building exits. (Section 10.1.7)

APPENDIX H TO CHAPTER 6
HAZUS AEBM REGULATIONS

6-A1 HAZUS AEBM Technology. The Federal Emergency Management Agency (FEMA)/National Institute of Building Sciences (NIBS) Multi-Hazard Loss Estimation Technology (HAZUS-MH MR2) and, specifically, the HAZUS Advanced Engineering Building Module (AEBM) are used by the Office with building-specific parameters, described in this appendix, to evaluate the Probability of Collapse of SPC-1 buildings.

6-A2 Probability of Collapse. The Probability of Collapse, P[COL], is calculated by Equation (A6-1):

$$P[COL] = P[COL|STR_5] \times P[STR_5] \qquad \text{(A6-1)}$$

where:

$P[COL|STR_5]$ = collapse factor of the HAZUS AEBM, as modified herein, and

$P[STR_5]$ = probability of Complete Structural Damage, based on HAZUS AEBM methods and parameters, as modified herein.

6-A3 Building-Specific Properties. Building-specific properties are based on the building type (structural system), or Model Building Type (MBT), building height (number of stories above seismic base), building age (pre-1933, 1933 – 1961 or post-1961 design vintage), availability of materials testing data, and Significant Structural Deficiencies.

Table A6-1 lists Significant Structural Deficiencies. Table A6-1 includes older buildings (pre-1933 buildings) and buildings that do not have available materials test data, and treats these conditions as Significant Structural Deficiencies.

SPC-1 buildings with no Significant Structural Deficiencies are evaluated using "Baseline" values of building-specific properties. SPC-1 buildings with one or more Significant Structural Deficiencies are evaluated using Sub-Baseline (SubBase), or Ultra-Sub-Baseline (USB) building-specific properties, as specified in Table A6-1.

Building-specific properties include parameters related to (1) building capacity, (2) building response, (3) Complete Structural Damage, and (4) building collapse. Appendix H Sections 6-A4 through 6-A7, define the parameters of interest related to building capacity, building response, Complete Structural Damage and building collapse, respectively, and specify appropriate values of these parameters.

6-A4. Building Capacity. Building-specific capacity properties of interest include the yield capacity control point (D_y, A_y) and the ultimate capacity control point (D_u, A_u), as calculated by Equations (A6-2 through A6-5, respectively):

$$A_y = C_s \cdot \gamma / \alpha_1 \qquad \text{(A6-2)}$$

$$D_y = 9.8 \cdot A_y \cdot T_e^2 \qquad \text{(A6-3)}$$

$$A_u = \lambda \cdot A_y \qquad \text{(A6-4)}$$

$$D_u = \lambda \cdot \mu \cdot D_y \qquad \text{(A6-5)}$$

where:

C_s = seismic design coefficient — values of C_s are given in Tables A6-2a and A6-2b, respectively,

α_1 = modal weight factor, Alpha 1 — values of α_1 are given in Table A6-4,

T_e = elastic period, in seconds — values of T_e are given in Table A6-3,

γ = yield strength factor, Gamma — values of γ are given in Table A6-5,

λ = "overstrength" factor, Lambda — values of λ are given in Table A6-5, and

μ = "ductility" factor, Mu — values of μ are given in Table A6-6.

6-A5 Building Response. Building-specific response parameters of interest include the elastic damping factor, β_E, and the degradation factor, Kappa. Values of β_E are given in Table A6-7 and values of the Kappa factor are given in Table A6-8.

6-A-6 Complete Structural Damage. Building-specific damage parameters of interest include the median spectral displacement of the Complete Structural Damage state, S_{dC}, and the associated lognormal standard deviation (Beta) factor, β_C. Values of β_C are given in Table A6-11. Median spectral displacement at the Complete Structural Damage state, S_{dC}, is calculated using Equation (A6-6):

$$S_{d,C} = \Delta_C \cdot H_R \cdot \alpha_2 / \alpha_3 \qquad \text{(A6-6)}$$

where:

Δ_C = interstory drift ratio (of the story with maximum drift) at the threshold of Complete Structural Damage — values of Δ_C are given in Table A6-9,

H_R = height of building at the roof level, in inches — default values of H_R are given in Table A6-3 as a function of the number of stories above grade,

α_2 = modal height factor, Alpha 2 — values of α_2 are given in Table A6-4, and

α_3 = modal shape factor, Alpha 3, relating maximum-story drift and roof drift, values of α_3 are given in Table A6-10.

6-A-7 Building Collapse. Building-specific values of the collapse factor, $P[COL|STR_5]$, that describe the fraction of the building likely to be collapsed given that the building has reached the Complete Structural Damage state, STR_5, are given in Table A6-12.

TABLE A6-1—SIGNIFICANT STRUCTURAL DEFICIENCY MATRIX

Significant Structural Deficiency/Condition[1]	Capacity		Response		Complete Structural Damage State						Collapse	
	Over-Strength		Duration		Fragility Curve Median[4]				Fragility Curve Variability - Beta Factor (β_C)		Collapse Factor (P[COL|STR$_5$])	
	Gamma and Lambda Factors		Degradation (Kappa) Factor		Maximum Story Drift Ratio (Δ_C)		Mode Shape (Alpha 3) Factor					
	SubBase	USB	SubBase	USB[5]	SubBase	USB	SubBase	USB[6]	SubBase	USB[5]	SubBase	USB[6]
Age (Pre-1933 buildings)	X	X[7]										
Materials Testing (None)	X								X			
No Redundancy									X		X	X[6]
Weak Story Irregularity					X		X	X[6]			X	X[6]
Soft Story Irregularity					X		X	X[6]			X	X[6]
Mass Irregularity					X							
Vertical Discontinuity	X				X							
Torsional Irregularity						X					X	X[6]
Deflection Incompatibilty[2]					X				X		X	X[6]
Short Column[3]	X					X						
Wood Deterioration		X	X									
Steel Deterioration		X	X									
Concrete Deterioration		X	X									
Weak Column-Steel	X				X							
Weak Column-Concrete	X		X		X							
No Cripple Wall Bracing					X		X	X[6]			X	X[6]
Topping Slab	X		X						X		X	X[6]
Inadequate Wall Anchorage		X							X			

1. Sub-Baseline (SubBase) and Ultra-Sub-Baseline (USB) properties are based on one, or more, significant structural deficiencies.
2. The Deflection Incompatibility structural deficiency applies only to concrete systems (C1, C2 and C3).
3. The Short Column structural deficiency applies only to concrete and masonry systems (C1, C2, C3, RM1 and RM2).
4. Effects of deficiencies related to drift and mode shape limited to a combined factor of 5 reduction in Complete median (of HAZUS default value).
5. Grey shading indicates USB performance is not defined/used for deficiencies related to degradation (kappa) and fragility curve (beta) factors.
6. USB performance required for systems with multiple, SubBase deficiencies related to either the mode shape (Alpha 3) factor or the collapse rate.
7. USB performance required for pre-1933 buildings with other over-strength-related deficiencies (else use SubBase performance for pre-1933 buildings).

TABLE A6-2a—SEISMIC DESIGN COEFFICIENT, Cs UBC SEISMIC ZONE 4

No. of Stories	Seismic Design Coefficient, C_s - UBC Seismic Zone 4 Locations (Zone 3 of older editions of the UBC)					
	Structural System (MBT)					
	S1 and C1		S2, S3, S4, S5, C2 and C3 (MH)		W1, W2, PC1, PC2, RM1, RM2, URM	
	Post-61	Pre-61	Post-61	Pre-61	Post-61	Pre-61
1	0.072	0.109	0.100	0.109	0.133	0.109
2	0.057	0.092	0.100	0.092	0.133	0.092
3	0.050	0.080	0.086	0.080	0.114	0.080
4	0.045	0.071	0.078	0.071	0.104	0.071
5	0.042	0.063	0.073	0.063	0.098	0.063
6	0.040	0.057	0.069	0.057	0.092	0.057
7	0.038	0.052	0.066	0.052	0.088	0.052
8	0.036	0.048	0.064	0.048	0.085	0.048
9	0.035	0.044	0.062	0.044	0.082	0.044
10	0.034	0.041	0.060	0.041	0.080	0.041
11	0.032	0.039	0.058	0.039	0.078	0.039
12	0.032	0.036	0.057	0.036	0.076	0.036
13	0.031	0.034	0.056	0.034	0.074	0.034
14	0.030	0.032	0.055	0.032	0.073	0.032
15	0.029	0.031	0.054	0.031	0.072	0.031
16	0.029	0.029	0.053	0.029	0.070	0.029
17	0.028	0.028	0.052	0.028	0.069	0.028
18	0.028	0.027	0.051	0.027	0.068	0.027
19	0.027	0.026	0.051	0.026	0.067	0.026
>= 20	0.027	0.024	0.050	0.024	0.067	0.024

TABLE A6-2b—SEISMIC DESIGN COEFFICIENT, Cs UBC SEISMIC ZONE 3

No. of Stories	Seismic Design Coefficient, C_s - UBC Seismic Zone 3 Locations (Zone 2 - older editions of the UBC)					
	Structural System (MBT)					
	S1 and C1		S2, S3, S4, S5, C2 and C3 (MH)		W1, W2, PC1, PC2, RM1, RM2, URM	
	Post-61	Pre-61	Post-61	Pre-61	Post-61	Pre-61
1	0.036	0.055	0.050	0.055	0.066	0.055
2	0.028	0.046	0.050	0.046	0.066	0.046
3	0.025	0.040	0.043	0.040	0.057	0.040
4	0.023	0.035	0.039	0.035	0.052	0.035
5	0.021	0.032	0.037	0.032	0.049	0.032
6	0.020	0.029	0.035	0.029	0.046	0.029
7	0.019	0.026	0.033	0.026	0.044	0.026
8	0.018	0.024	0.032	0.024	0.043	0.024
9	0.017	0.022	0.031	0.022	0.041	0.022
10	0.017	0.021	0.030	0.021	0.040	0.021
11	0.016	0.019	0.029	0.019	0.039	0.019
12	0.016	0.018	0.029	0.018	0.038	0.018
13	0.015	0.017	0.028	0.017	0.037	0.017
14	0.015	0.016	0.027	0.016	0.036	0.016
15	0.015	0.015	0.027	0.015	0.036	0.015
16	0.014	0.015	0.026	0.015	0.035	0.015
17	0.014	0.014	0.026	0.014	0.035	0.014
18	0.014	0.013	0.026	0.013	0.034	0.013
19	0.014	0.013	0.025	0.013	0.034	0.013
>= 20	0.013	0.012	0.025	0.012	0.033	0.012

TABLE A6-3—DEFAULT BUILDING HEIGHTS AND ELASTIC PERIODS

No. of Stories	Default Building Height, H_R, and Elastic Period, T_e, Properties													
	Structural System (MBT)													
	W1 and W2 (MH)		S1		C1		S2		S4 and S5		C2, C3, PC2, RM1, RM2, URM		S3 and PC1	
	H_R (ft)	T_e (sec)	H_R (ft)	T_e (sec)	H_R (ft)	T_e (sec)	H_R (ft)	T_e (sec)	H_R (ft)	T_e (sec)	H_R (ft)	T_e (sec)	H_R (ft)	T_e (sec)
1	14	0.35	14	0.40	12	0.40	14	0.40	14	0.35	12	0.35	15	0.35
2	24	0.38	24	0.50	20	0.40	24	0.43	24	0.35	20	0.35	25	0.39
3	34	0.49	36	0.69	30	0.48	36	0.59	36	0.44	30	0.39	35	0.50
4	44	0.60	48	0.87	40	0.62	48	0.73	48	0.55	40	0.48		
5	54	0.70	60	1.04	50	0.76	60	0.86	60	0.65	50	0.57		
6			72	1.20	60	0.89	72	0.99	72	0.74	60	0.65		
7			84	1.36	70	1.03	84	1.11	84	0.84	70	0.73		
8			96	1.51	80	1.16	96	1.22	96	0.92	80	0.81		
9			108	1.66	90	1.29	108	1.34	108	1.01	90	0.88		
10			120	1.81	100	1.41	120	1.45	120	1.09	100	0.95		
11			132	1.95	110	1.54	132	1.55	132	1.17	110	1.02		
12			144	2.09	120	1.67	144	1.66	144	1.25	120	1.09		
13			156	2.23	130	1.79	156	1.76	156	1.33	130	1.16		
14			168	2.36	140	1.91	168	1.86	168	1.40	140	1.23		
15			180	2.50	150	2.04	180	1.96	180	1.48	150	1.29		
16			192	2.63	160	2.16	192	2.06	192	1.55	160	1.35		
17			204	2.76	170	2.28	204	2.15	204	1.62	170	1.42		
18			216	2.89	180	2.40	216	2.25	216	1.70	180	1.48		
19			228	3.02	190	2.52	228	2.34	228	1.77	190	1.54		
>= 20			240	3.14	200	2.64	240	2.43	240	1.84	200	1.60		

TABLE A6-4—ALPHA 1 AND ALPHA 2, MODAL FACTORS

No. of Stories	Alpha 1 (α_1) - Modal Weight Factor				Alpha 2 (α_2) - Modal Height Factor	
	Structural System (MBT)				Structural System (MBT)	
	S1 and C1	W1, W2, S2, S3, S4, C2, C3, PC2, RM1 and RM2	PC1 and URM	MH	MH	All Systems (except MH)
1	0.75	0.8	0.75	1.00	1.00	0.75
2	0.75	0.8	0.75			0.75
3	0.75	0.8	0.75			0.75
4	0.75	0.8				0.75
5	0.75	0.8				0.75
6	0.73	0.79				0.72
7	0.71	0.78				0.69
8	0.69	0.77				0.66
9	0.67	0.76				0.63
10	0.65	0.75				0.60
11	0.65	0.75				0.60
12	0.65	0.75				0.60
13	0.65	0.75				0.60
14	0.65	0.75				0.60
>= 15	0.65	0.75				0.60

TABLE A6-5—LAMBDA FACTOR

No. of Stories	Gamma Factor (γ)	Lambda Factor (λ)														
		Baseline Performance					SubBase Performance					USB Performance				
		Structural System (MBT)					Structural System (MBT)					Structural System (MBT)				
		W1, S1, C1	W2, C2	S4, C3	Other MBT	PC1, URM	W1, S1, C1	W2, C2	S4, C3	Other MBT	PC1, URM	W1, S1, C1	W2, C2	S4, C3	Other MBT	PC1, URM
1	2.70	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
2	2.50	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
3	2.25	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
4	2.00	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
5	1.88	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
6	1.80	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
7	1.75	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
8	1.71	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
9	1.69	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
10	1.67	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
11	1.65	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
12	1.65	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
13	1.65	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
14	1.65	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17
>= 15	1.65	2.00	2.00	1.83	1.67	1.33	1.75	1.75	1.63	1.50	1.25	1.50	1.50	1.42	1.33	1.17

TABLE A6-6—DUCTILITY FACTOR *Mu*

No. of Stories	*Mu* (μ) Factor (All Systems)
1	6.00
2	6.00
3	4.94
4	4.41
5	4.07
6	3.82
7	3.63
8	3.48
9	3.35
10	3.24
11	3.15
12	3.07
13	3.00
14	3.00
>= 15	3.00

TABLE A6-7—ELASTIC DAMPING

Structural System (MBT)	β_E Elastic Damping (% of Critical)
S1, S2, S3 and S4	5
C1, C2, PC1 and PC2	7
RM1 and RM2	7
C3 and S5	7
W1 and W2	10

TABLE A6-8—DEGRADATION KAPPA FACTORS

Scenario Earthquake Criteria		Degradation (Kappa) Factors - (κ_S, κ_M and κ_L)			
Minimum Distance Site to Fault[1] (km)	Maximum Magnitude[2]	Baseline Performance		SubBase Performance	
		Post-61	Pre-1961	Post-61	Pre-1961
< 5	All	0.8	0.7	0.6	0.5
5 - 10	M_{max} <= 6.5	0.8	0.7	0.6	0.5
5 - 10	M_{max} > 6.5	0.7	0.6	0.5	0.4
10 - 25	M_{max} <= 6.5	0.7	0.6	0.5	0.4
10 - 25	7.0 >= M_{max} > 6.5	0.6	0.5	0.4	0.3
10 - 25	M_{max} > 7.0	0.5	0.4	0.3	0.2
25 - 50	M_{max} <= 7.0	0.5	0.4	0.3	0.2
25 - 50	M_{max} > 7.0	0.4	0.3	0.2	0.1
> 50	All	0.4	0.3	0.2	0.1

1. Minimum distance to the fault that controls 1-second period ground motions at the building site.
2. Maximum magnitude (Mmax) of fault that controls 1-second ground motions at the building site

TABLE A6-9—INTERSTORY DRIFT RATIO — MEDIAN COMPLETE STRUCTURAL DAMAGE

Structural System (MBT)	Interstory Drift Ratio (max story) - Median Complete Structural Damage (Δ_C)					
	Baseline Performance		SubBase Performance		USB Performance	
	Post-61	Pre-61	Post-61	Pre-61	Post-61	Pre-61
W1, W2 (MH)	0.075	0.075	0.060	0.060	0.038	0.038
S1, C1, S2 and C2	0.060	0.050	0.050	0.040	0.030	0.025
S3, S4, PC1, PC2, RM1 and RM2	0.053	0.044	0.044	0.035	0.027	0.022
S5, C3 and URM		0.035		0.028		0.018

TABLE A6-10—ALPHA 3 (À3) MODAL SHAPE FACTOR

No. of Stories	Alpha 3 (α_3) Modal Shape Factor - Ratio of Maximum Interstory Drift to Average Interstory Drift								
	When Combined with Baseline Interstory Drift Ratios (Table A6-9)			When Combined with SubBase Interstory Drift Ratios (Table A6-9)			When Combined with USB Interstory Drift Ratios (Table A6-9)		
	Baseline Performance	SubBase Performance	USB Performance	Baseline Performance	SubBase Performance	USB Performance	Baseline Performance	SubBase Performance	USB Performance
1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2	1.21	1.62	2.03	1.21	1.62	2.03	1.21	1.62	2.03
3	1.35	2.04	2.73	1.35	2.04	2.73	1.35	2.04	2.50
4	1.45	2.36	3.27	1.45	2.36	3.27	1.45	2.36	2.50
5	1.54	2.63	3.72	1.54	2.63	3.72	1.54	2.50	2.50
6	1.62	2.87	4.11	1.62	2.87	4.00	1.62	2.50	2.50
7	1.69	3.07	4.46	1.69	3.07	4.00	1.69	2.50	2.50
8	1.75	3.26	4.77	1.75	3.26	4.00	1.75	2.50	2.50
9	1.81	3.43	5.00	1.81	3.43	4.00	1.81	2.50	2.50
10	1.86	3.59	5.00	1.86	3.59	4.00	1.86	2.50	2.50
11	1.91	3.73	5.00	1.91	3.73	4.00	1.91	2.50	2.50
12	1.96	3.87	5.00	1.96	3.87	4.00	1.96	2.50	2.50
13	2.00	4.00	5.00	2.00	4.00	4.00	2.00	2.50	2.50
14	2.04	4.12	5.00	2.04	4.00	4.00	2.04	2.50	2.50
>= 15	2.08	4.23	5.00	2.08	4.00	4.00	2.08	2.50	2.50

TABLE A6-11—LOGNORMAL STANDARD DEVIATION (BETA) VALUES — COMPLETE STRUCTURAL DAMAGE

No. of Stories	Lognormal Standard Deviation (Beta) Values - Complete Structural Damage (β_C)			
	Baseline Performance		SubBase Performance	
	Post-61	Pre-61	Post-61	Pre-61
1	0.85	0.90	0.95	1.00
2	0.85	0.90	0.95	1.00
3	0.85	0.90	0.95	1.00
4	0.84	0.89	0.94	0.99
5	0.83	0.88	0.93	0.98
6	0.82	0.87	0.92	0.97
7	0.81	0.86	0.91	0.96
8	0.80	0.85	0.90	0.95
9	0.79	0.84	0.89	0.94
10	0.78	0.83	0.88	0.93
11	0.77	0.82	0.87	0.92
12	0.76	0.81	0.86	0.91
13	0.75	0.80	0.85	0.90
14	0.75	0.80	0.85	0.90
>= 15	0.75	0.80	0.85	0.90

TABLE A6-12—COLLAPSE FACTOR

Structural System (MBT)	Collapse Factor - Likelihood of Collapse given Complete Structural Damage - P[COL|STR_5]		
	Baseline Performance	SubBase Performance	USB Performance
W1 and W2	0.05	0.10	0.20
S1, S2, S3, S4 and S5	0.08	0.15	0.30
C1, C2 and C3	0.13	0.25	0.50
RM1 and RM2	0.13	0.25	0.50
PC1 and PC2	0.15	0.30	0.60

HISTORY NOTE APPENDIX FOR CHAPTER 6

Administrative Regulations for the Office of Statewide Health Planning and Development (Title 24, Part 1, California Code of Regulations)

The format of the history notes has been changed to be consistent with the other parts of the *California Building Standards Code*. The history notes for prior changes remain within the text of this code.

1. (OSHPD 1/96) Adoption of Chapter 6, Seismic Evaluation Procedures for Hospital Buildings, Part 1, Title 24, C.C.R. Filed with the secretary of state on April 8, 1997, effective April 8, 1997. Approved by the California Building Standards Commission on February 6, 1997.

2. (OSHPD 1/97) New Article 1-Definitions and Requirements based on SB 1953. Approved by the California Building Standards Commission on March 18, 1998. Filed with the Secretary of State on March 25, 1998, effective March 25, 1998.

3. (BSC 2/99) Article 1-7, Conflict of Interest Code. Amend Section 1-701. Approved by the Fair Political Practices Committee on October 29, 1999. Filed with the Secretary of State on December 31, 1999, effective January 30, 2000.

4. (OSHPD EF 1/00) Part 1, Chapter 6, Articles 1, 10, 11 and Appendix. Approved as submitted by the California Building Standards Commission on February 28, 2000. Filed with the Secretary of State on March 3, 2000, effective March 3, 2000. Permanent approval by California Building Standards Commission on May 24, 2000. Certification of Compliance filed with Secretary of State May 26, 2000.

5. (OSHPD EF 2/00) Part 1, Amend Chapter 6, Articles 1, 2, 10 and 11. Emergency approval by the California Building Standards Commission on May 24, 2000. Filed with the Secretary of State on May 26, 2000, effective May 26, 2000. Permanent approval by California Building Standards Commission September 20, 2000. Certification of Compliance filed with Secretary of State November 15, 2000.

6. (OSHPD EF 5/01) Emergency adoption of amendments to hospital seismic safety evaluation regulations contained in Title 24, C.C.R., Part 1, Chapter 6. Approved by the California Building Standards Commission on November 28, 2001. Filed with the Secretary of State on December 4, 2001, effective December 4, 2001.

7. (OSHPD EF 01/02) Amend Chapter 6 and 7 of Part 1. Approved as emergency by the California Building Standards Commission on January 15, 2003, and filed with the Secretary of State on January 16, 2003. Effective January 16, 2003.

8. (OSHPD EF 01/02) Amend Chapters 6 and 7 of Part 1. Approved as permanent emergency by the California Building Standards Commission. Permanent approval on May 14, 2003. Certification of Compliance filed with the Secretary of State on May 15, 2003. Effective January 16, 2003.

9. (OSHPD EF 01/05) Amend Part 1, Chapter 6, Article 11 and Table 11.1. Approved as emergency by the California Building Standards Commission on December 13, 2005. Filed with the Secretary of State on December 14, 2005 with an effective date of December 14, 2005.

10. (OSHPD EF 01/05) Amend Part 1, Chapter 6, Article 11 and Table 11.1. Re-adopted/approved as emergency by the California Building Standards Commission on March 22, 2006. Filed with the Secretary of State on March 30, 2006 with an effective date of March 30, 2006.

11. (OSHPD 01/04) Amend Article 1 for nonconforming hospital buildings. Filed with Secretary of State on May 23, 2006, and effective on the 30th day after filing with the Secretary of State.

12. (OSHPD EF 01/05) Amend Title 24, Part 1, Chapter 6, Article 11 and Table 11.1. The language for the permanent rule will remain effective and unchanged from the readoption/approval of Emergency Finding (OSHPD EF 01/05) Supplement dated May 30, 2006. Approved as permanent by the California Building Standards Commission on July 27, 2006 and filed with the Secretary of State on July 28, 2006.

13. (OSHPD EF 01/07) Amend Title 24, Part 1, Chapter 6, Article 1, Article 2, Article 4, Article 6, Article 11, Table 11.1. Approved by the California Building Standards Commission on July 19, 2007. Filed with the Secretary of State July 20, 2007, effective January 1, 2008.

14. (OSHPD EF 01-07) Amend Title 24, Part 1, Chapter 6, Article 1, Article 2, Article 4, Article 6, Article 11 and Table 11.1. Approved by the California Building Standards Commission on July 19, 2007. Filed with the Secretary of State on July 20, 2007, effective January 1, 2008. It was approved as permanent by the California Building Standards Commission on May 21, 2008 and filed with the Secretary of State on May 23, 2008.

15. (OSHPD EF 02/07) Amend Title 24, Part 1, Chapter 6, definitions added and Chapter amended throughout with a new Appendix H to Chapter 6. Approved as an emergency regulation by the California Building Standards Commission on November 14, 2007, filed with the Secretary of State on November 29, 2007. Effective November 29, 2007. It was approved as permanent by the California Building Standards Commission on May 21, 2008 and filed with the Secretary of State on May 23, 2008.