FROM: HQ AFCESA/CES
139 Barnes Drive, Suite 1
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SUBJECT: **Engineering Technical Letter (ETL) 97-10: Structural Evaluation of Existing Buildings for Seismic and Wind Loads**

1. Purpose. Instructions for evaluating the structural vulnerability of existing Air Force buildings subject to loads from earthquake and high wind are included in this ETL. Recommendations for establishing preliminary definitions of structural modifications of real property required for the mitigation of risk to human life and operational readiness are provided. Compliance with this ETL satisfies the minimum provisions of the *Standards of Seismic Safety for Existing Federally Owned or Leased Buildings* (Reference 1). Evaluation statements for buildings in high wind exposure areas are also included.

2. Application. This ETL applies to all Air Force installations. This document is an aid to assist the practicing engineer to quickly evaluate a building identified as a potential life-safety risk for the performance objective regarding damage control or post-event operations. Each base (or installation) will perform evaluations of buildings according to the following criteria:

- Buildings determined to be a seismic risk in accordance with ETL 93-3 (Reference 2) will be structurally evaluated for seismic hazards using the procedures of this ETL.
- Buildings located in high wind regions, as defined in paragraph 4.11 below, will be structurally evaluated for wind/hurricane hazards using the procedures of this ETL.

Buildings which are exposed to both seismic loads and wind are evaluated for both events.

2.1. Authority: Executive Order 12941, *Seismic Safety of Existing Federally Owned or Leased Buildings*, 1 December 1994.

2.2. Effective Date: Immediately. Expires five years from date of issue.

3. References. References are provided in Attachment 1.

4. Definitions.

4.1. *Building*: Any structure, fully or partially enclosed, used or intended for sheltering people or property. Does not include bridges, transmission towers, industrial towers and equipment, or hydraulic structures.

4.2. *Hazards*: A source of danger with potential to cause illness, injury, or death to people or damage to a facility or the environment.

4.3. *Seismic Risk Inventory*: An inventory of all buildings under the jurisdiction of an agency which is categorized to determine each building's relative probability of presenting an unacceptable seismic risk to life-safety.

4.4. *Evaluation*: A procedure to determine whether life-safety risks exist in a building.

4.5. *Mitigation*: The substantial reduction of life-safety risk from seismic or wind hazards involving a building and/or building site. Examples include demolition, permanent evacuation, change in occupancy, and rehabilitation.

4.6. *National Design Force Exceedance Factor (NDFEF)*: Ratio of demands of the large but rare earthquake (with 2500-year return period) to the building code level design earthquake (with 500-year return period). The ratio is formed using spectral accelerations in the short-period range.

4.7. *Structural*: The portions of a building that hold it up and resist gravity, earthquakes, wind, and other types of loads. Examples include columns; beams; floor or roof sheathing; slabs or decking; and load-bearing walls.

4.8. *Nonstructural*. The portions of a building that include every part of it and all of its contents with the exception of the structure. They include electrical, mechanical, and architectural elements. Common nonstructural items include ceilings, windows, office equipment, computers, non-bearing interior partition walls, and exterior wall panels.

4.9. *Risk*: The quantitative or qualitative expression of possible loss that considers both the probability that a hazard will cause harm and the consequences of that event.

4.10. *Building Configuration*: Either regular or irregular. Regular configurations have uniform proportions which retain a compact architectural form. Irregular configurations include abrupt changes in geometric shape (either plan or profile), large in plan, excessive length-to-width proportion or center of mass not coincident with center of resistance. Irregularity does include interruption of structural elements and abrupt changes in lateral stiffness. Large plan buildings, such as aircraft hangers or warehousing, are irregular because the building will be responding to different modes because of the long spans used in the construction. The more significant of the irregular configurations includes the "soft story" (ground-level story is less stiff than those above), discontinuous shear walls (location different between floors), and reentrant corners (L-shape).

4.11. *High Wind Regions*: Those regions where the basic wind speed, V, as determined from Figure 6-1, ASCE 7-95 (Reference 3), is greater than or equal to 177 km/h (110 mph) or the installation is located within 161 kilometers (100 miles) of the hurricane oceanline. These hurricane-prone regions include areas vulnerable to hurricanes, such as the U.S. Atlantic and Gulf Coasts, Hawaii, Puerto Rico, Guam, Virgin Islands, and American Samoa as defined in Table 6-4, ASCE 7-95 (Reference 3). In overseas regions, high wind regions shall include installations with comparable wind or hurricane hazards based upon regional design criteria or climatic data.

5. Specific Requirements.

5.1. Basic Program Requirements. The *Standards of Seismic Safety for Existing Federally Owned or Leased Buildings* and *Commentary* (ICSSC Standards) (Reference 1) has been developed by the Interagency Committee on Seismic Safety in Construction (ICSSC) in conjunction with the National Institute of Standards and Technology (NIST). The ICSSC Standards provide minimum standards for the evaluation and mitigation of seismic hazards in existing buildings. There are four compliance categories that must be satisfied: (1) structural, (2) nonstructural, (3) geologic/site hazards, and (4) adjacency. The Air Force considers that compliance with this ETL satisfies compliance categories (1) structural and (4) adjacency of the ICSSC Standards. Compliance in categories (2) nonstructural and (3) geologic/site hazards are determined using procedures independent of this document.

5.1.1. The minimum performance objective of the Standards is to achieve Substantial Life-Safety in Federal buildings. This is the performance objective where the earthquake may cause significant building damage that may not be repairable, though it is not expected to significantly jeopardize life from structural collapse, falling hazards or blocked routes of entrance or egress. Compliance with FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (Reference 4), is assumed to achieve this level of performance.

5.1.2. Program compliance is assumed if a building is exempted by the ICSSC Standards, determined by evaluation to comply with the ICSSC Standards, or unacceptable seismic risks are completely mitigated. The compliance determinations apply to high wind risk areas.

5.2. Special Considerations and Higher Performance Objectives. FEMA 178 prescribes the minimum standard for building evaluations required by this ETL. As permitted by the ICSSC Standards, better performance standards are prescribed by this ETL for the Air Force to meet its post-emergency mission and in consideration of other important reasons. The added considerations include the following:

5.2.1. National Differences in Seismicity of Moderate and Large Earthquakes. Particularly in the central and eastern United States, the ratio of the large but rare earthquake to the building code design level earthquake (termed the National Design Force Exceedance Factor, NDFEF) is much greater than in certain regions of the western United States where damaging earthquakes have occurred and influenced seismic building development (Reference 5). Thus, when designing for the often lower seismic code design forces in the regions of higher earthquake NDFEF values, there is less certainty on how the building will perform under the large earthquake. As in building code design, the focus of the ICSSC Standards and FEMA 178 is on life-safety and non-collapse in the event of a large earthquake. Hence, this ETL prescribes an additional performance standard requiring a verification for life-safety performance for the large earthquake when the NDFEF exceeds certain limits.

5.2.2. Reconsideration of Exempted Post-Benchmark Buildings. In the screening process, certain buildings were exempted from structural evaluation according to criteria provided by ETL 93-3 (Reference 2). Included in the exempted buildings were post-benchmark buildings. These are buildings that were designed and built after the adoption of seismic code provisions which have been generally considered to provide acceptable life-safety protection. As discussed in paragraph 5.2.1, this presumption of acceptability is inappropriate in certain regions where the peak ground accelerations of large and moderate earthquakes vary by a relatively large margin. Consequently, exempted post-benchmark buildings which are Immediate Occupancy or High Risk (Category I or III) located in regions with the National Design Force Exceedance Factor (NDFEF) greater than 1.5 will be returned to the list of buildings within the seismic risk inventory designated for structural evaluation.

5.2.3. Performance Objectives Beyond Substantial Life Safety. To acceptably address various mission demands, additional performance requirements are prescribed by this ETL. They are identified as performance goals in *Seismic Design for Essential Buildings* (Reference 6). They are specified in terms of four earthquake levels; EQ-Y, EQ-I, EQ-II, EQ-III. EQ-Y is the earthquake in which the structure reaches the yield limit and remains essentially or nearly elastic. EQ-Y is defined by the capacity of the structure, whereas EQ-I, EQ-II and EQ-III are defined by the probability of their occurrence, as stated below. It should be noted that an earthquake of any particular return period could occur at any time or frequency. For example, three large earthquakes (Modified Mercalli Intensity values in excess of VIII) occurred within a two-year time span (1811-12) in New Madrid, Missouri.

5.2.3.1. EQ-Y is not used for the structural evaluation of existing buildings. EQ-Y is used to perform nonstructural evaluations.

5.2.3.2. EQ-I has a return period of 70 years. This is the nominal value that corresponds to an event that has a 50 percent probability of being exceeded in 50 years.

5.2.3.3. EQ-II has a return period of 1000 years for the occupancy categories of Essential Facilities (I) and Hazardous Facilities (II) and a return period of 500 years for the occupancy categories of Special Occupancy (III) and Standard Occupancy (IV). It should be noted that the 475-year event is the normal return period for an event that has a 10 percent probability of being exceeded in 50 years and is the basis for *Seismic Design for Buildings* (Reference 7).

5.2.3.4. EQ-III has a return period of 4000 years for occupancy categories I and II and a return period of 2500 years for occupancy categories III and IV. EQ-III is only used when NDFEF exceeds 1.5.

5.2.3.5. Table 1 describes the adopted four performance goals (A, B, C, D) under column headings of the applicable performance objective and the associated earthquake level used in determining compliance with the particular performance criterion. Table 2 provides the performance requirements for earthquakes and associated earthquake return periods for each of the ETL 93-3 performance categories. Note that Performance Category I, *Immediate Occupancy*, of ETL 93-3 includes the Hazardous Facilities category of *Seismic Design for Buildings* (Reference 7). Performance objective C3 is the basis for the *Seismic Design for Buildings* design earthquake. (Note: the actual return period is 475 years; the 500-year event has approximately 9.5% probability of being exceeded in 50 years.)

Table 1. Performance Goals

<table>
<tr><td>Validation Earthquake</td><td colspan="2">EQ-Y</td><td colspan="2">EQ-I</td><td colspan="2">EQ-II</td><td colspan="2">EQ-III</td></tr>
<tr><td>Performance Objectives</td><td colspan="2">Fully Functional</td><td colspan="2">Immediate Occupancy</td><td colspan="2">Damage Control</td><td colspan="2">Survival</td></tr>
<tr><td rowspan="3">Performance Goals</td><td rowspan="3">A</td><td rowspan="3">Full Service</td><td rowspan="3">B</td><td rowspan="3">Immediate Occupancy Almost Full Service</td><td>C1</td><td>Damage Control</td><td rowspan="3">D</td><td rowspan="3">Substantial Life-Safety</td></tr>
<tr><td>C2</td><td>• Safe Exit
• Maintain Emergency Function</td></tr>
<tr><td>C3</td><td>• Containment
• Life-Safety</td></tr>
<tr><td>Deformation Range</td><td colspan="2">Up to Elastic Limit</td><td colspan="2">Elastic Limit to Major Yielding</td><td colspan="2">Major Yielding to Initial Deterioration</td><td colspan="2">Initial Deterioration Ultimate Limits</td></tr>
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Table 2. Performance Requirements for Seismic Loads

HQ AFCESA ETL 93-3 Performance Category	EQ-I		EQ-II		EQ-III		AFJMAN 32-1049 Category
	Performance Goal	Return Period	Performance Goal	Return Period	Performance Goal	Return Period	
I Immediate Occupancy	B	70	C2	1000	D	4000	I Essential Facilities
	N/A	N/A	C3	1000	D	4000	II Hazardous Facilities
III High Risk	N/A	N/A	C2	500	D	2500	III Special Occupancy
IV Other Buildings	N/A	N/A	C3	500	D	2500	IV Standard Occupancy
V Other Hazards (non structural only)							

5.2.4. New Generation USGS Ground Motion Hazard Maps. The evaluation procedures of this ETL incorporate new generation probabilistic ground motion hazard maps produced for the National Earthquake Hazard Reduction Program by the U.S. Geological Survey (USGS). The hazard maps are the spectral response acceleration maps (as a percent of gravity) at periods of 0.2 sec and 1.0 sec for a soil profile Type B. The mapped spectral response accelerations have a 10% chance of exceedance in 50 years and a 2% chance of exceedance in 50 years (which can also be expressed as a 10% chance of exceedance in 250 years). These probabilities correspond to shaking that is expected to occur, on the average, about once every 500 years (or more exactly, 474 years) and 2500 years. The 10%/50 year maps represent the conventional “design earthquake” as usually mapped in building code provisions. The 2%/50 year maps represent the rare, but possible, large earthquake referred to in the ETL as the Collapse Limit Earthquake (CLE) for non-essential buildings. It is the EQ-III earthquake with a 2500 year return period. The maps in Attachment 2 are used to determine the spectral acceleration values for the 70-, 500-, 1000-, 2500- and 4000-year earthquake return periods required by this ETL. Attachment 2 contains these spectral acceleration values in tabular form for each Air Force base within the continental United States. The table also includes the National Design Force Exceedance Factor, NDFEF, used to determine if the building performance must be verified for EQ-III in those cases where adequacy for EQ-II has been established. Attachment 2 includes instructions for determining appropriate spectral acceleration values at locations outside of the adjacent 48 states, for Air Reserve bases, and Air National Guard locations. Attachment 3 provides a description of earthquake effects on buildings and the representation of earthquake force demands in terms of spectral acceleration values from the new USGS ground motion hazard maps provided in Attachment 2.

5.2.5. AFJMAN 32-1049V2*, Seismic Design Guidelines for Essential Buildings* (Reference 9), uses dynamic analysis and multi-level ground motions for the design of earthquake resistant buildings. Both considerations are required for an adequate description of the performance levels required by this document. The FEMA 178 provisions provide only for the accommodation of building code seismic design force levels. Select provisions of AFJMAN 32-1049 are incorporated into the evaluation procedure described herein to evaluate applicable multi-level performance goals of Air Force buildings.

5.2.6. Consideration of Wind. High winds pose a serious threat to the many Air Force installations in hurricane exposure areas. The effects of Hurricane Hugo at Charleston AFB, and Hurricane Andrew at Homestead AFB are recent reminders of the threat to the Air Force operational readiness posture. The necessity to assess the vulnerability of buildings and the effect on forces and weapons is underscored by the recovery time experienced at both locations. The performance requirements for wind are set forth in Table 3. Wind effects and force demands are explained in Attachment 4.

Table 3. Performance Requirements for Wind

HQ AFCESA ETL 93-3 Performance Category	Wind		Importance Factor	Allowable Stress Increase	ASCE 7-95 Category	AFJMAN 32-1049 Category
	Performance Goal	Return Period				
I Immediate Occupancy	B	100	1.15	1/3 or (1)	IV	I Essential Facilities
	N/A	N/A	N/A	N/A	N/A	II Hazardous Facilities
III High Risk	C_2	100	1.15	(2)	III	III Special Occupancy
IV Other Buildings	C_3	50	1.00	(2)	II	IV Standard Occupancy
V Other Hazards (non struct-ural only)						

(1) Use LRFD or strength design load factors for Category I buildings.

(2) Increase allowable stress to nominal strength using factors for design strength as defined in FEMA 222, but without capacity reduction factors ($\phi = 1.0$).

Wood	-	2.0 times allowables
Steel	-	1.7 times allowables or LRFD w/o load factor on wind loads
Concrete	-	1.0 (strength design) w/o load factor on wind loads
Masonry	-	2.5 times allowables

5.2.7. Validation: Wind Controls Design**.** As stated in the Structural Engineer Association of California (SEAOC) *Recommended Lateral Force Requirements* and *Commentary* (Reference 10), though wind and seismic forces are both laterally applied to the structure, they are fundamentally different: Wind is an exterior surface applied force (unless the building envelope is breached), while seismic force is an inertially applied force. Furthermore, wind design provisions consider an elastic response of the structure, whereas seismic design forces are based on inelastic behavior in the structure, and the maximum expected earthquake forces can create several reversed cycles of inelastic deformations. Consequently, even when wind governs the stress or drift design, the resisting system must still conform to the special requirements for seismic systems to accommodate the inertial forces. As SEAOC observes, such requirements provide for the inelastic energy absorption required to resist potential seismic loadings expected to exceed the specified design forces. Accordingly, the guidelines of this ETL require an integrated natural hazard (wind and seismic) evaluation of the building's lateral load resisting structural system. For wind as well as earthquake controlled design, an EQ-III evaluation is conducted as required to establish the adequacy of the structural system for the inelastic energy absorption required to resist potential seismic loadings expected to exceed the specified design forces.

5.3. Evaluation and Upgrading: An Overview. The evaluation and upgrade procedure consists of six basic phases: inventory reduction and prioritization; evaluation and report programming; preliminary design; final design and preparation of contract documents; and construction. Figure 1 shows the basic phases and certain key steps within two of the phases. Especially in structural rehabilitation work, the conduct of each phase and its integration with the other phases are critical to the success of the overall facility mitigation project.

5.3.1. Those buildings considered a seismic risk are identified and prioritized in Phase I using the procedures described in Air Force ETL 93-3 (Reference 2). The inventory, screening, and prioritization are accomplished by Air Force people who work at the installations. Supplemental guidance for screening and prioritization of buildings considered high wind risk is provided in Attachment 4 and Attachment 7.

5.3.2. A structural engineer experienced in seismic design and the evaluation of buildings accomplishes the evaluation and report of Phase II. The objective of an evaluation is to quickly identify and screen out buildings which have a complete lateral-force-resisting system which satisfies certain minimum strength criteria. Those buildings which do not pass a rapid evaluation are looked at in more detail, depending upon the deficiencies determined by rapid procedures. The detailed evaluation will

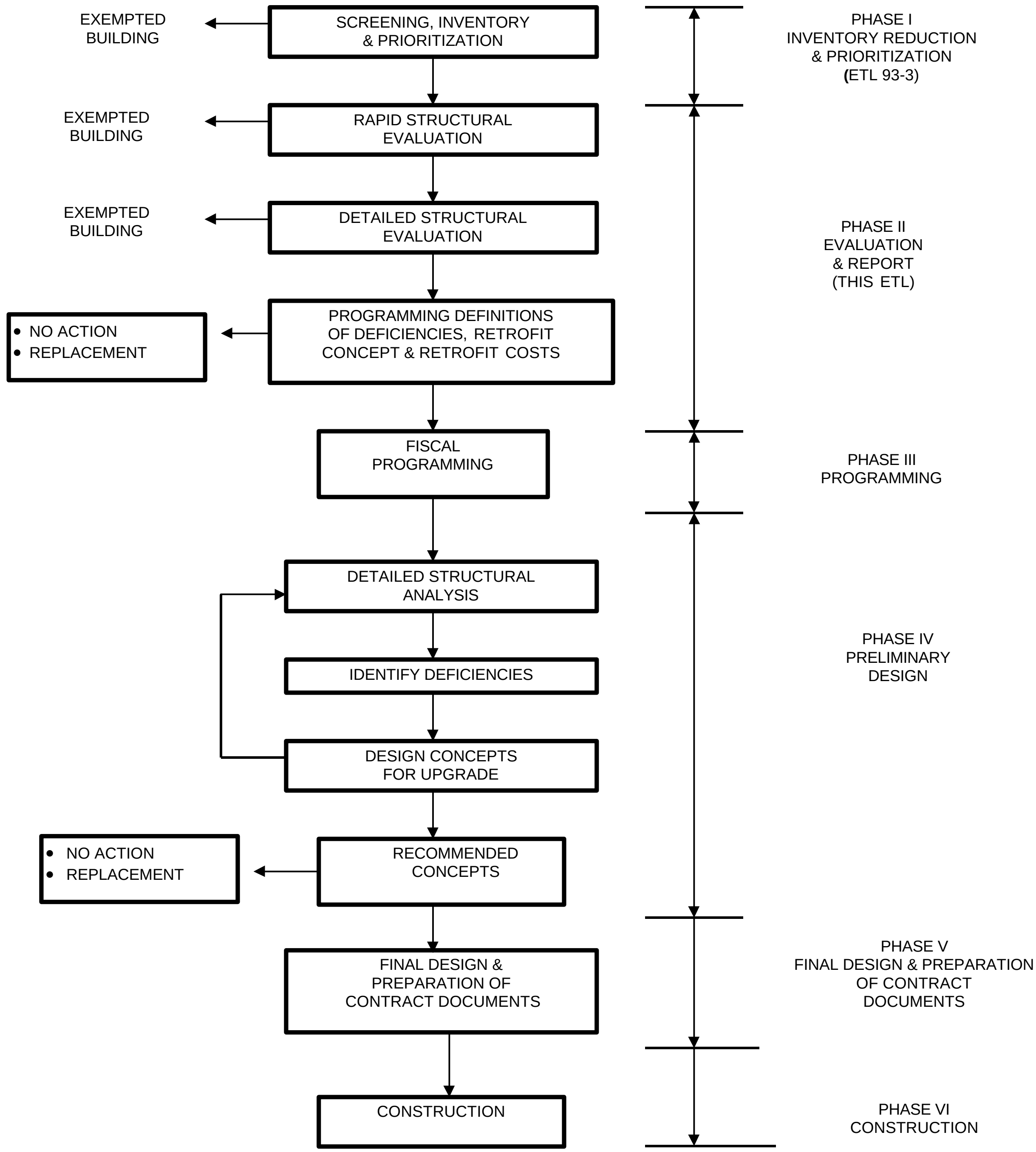

Figure 1. Evaluation and Upgrade Procedure

focus on those components of the building which fail the rapid evaluation. Using the deficiencies identified in either the rapid evaluation or the detailed evaluation, the evaluator prepares a schematic structural concept and first order cost estimate for programming of the rehabilitation work required. References 11, 12, and 13 provide some rehabilitation guidelines. Cost of rehabilitation may be approximated using cost data in FEMA 156 and FEMA 157 (References 14 and 15). The schematic concept may not be the final form of the rehabilitation scheme. The level of detail required to decide upon a rehabilitation scheme is accomplished in the design phase (Phase IV) of mitigation. Mitigation schemes other than rehabilitation may be recommended by the evaluator and include the following alternatives.

5.3.2.1. Building Replacement. Rehabilitation costs may be comparable to the construction of a replacement facility. In this case, consideration should be given to mitigation by constructing a new building conforming to current building code seismic design provisions.

5.3.2.2. Occupancy Reduction. A building may become acceptable without structural rehabilitation by reducing its occupancy risk level by changing its occupancy. An example is converting a building from functioning as a communications center to an office building, or a dormitory to a storage facility.

5.3.2.3. Abandonment and Relocation. The building may be "mothballed" or demolished.

5.3.3. Phase III will include seismic rehabilitation work prioritization and programming. Once questionable conditions in a building and/or its nonstructural elements have been identified, the seismic and/or wind rehabilitation work must be planned for accomplishment. All buildings which fail the seismic risk evaluation will be programmed for mitigation in competition with other base requirements using available programming and budgeting procedures. All buildings which fail the evaluation will remain on the seismic risk inventory until mitigation has been effected. The final decision of mitigation means will be made by the building owner (MAJCOM or base).

5.3.4. Phase IV requires the structural engineer to complete the preliminary design. More detailed structural analyses, site investigations, and any necessary material testing are conducted to determine more definitely the structural adequacy of the existing building or to identify possible additional deficiencies.

5.3.4.1. Alternative upgrading concepts will be developed. Each alternative upgrading concept will be evaluated for compliance with the acceptance criteria. Reanalysis of each concept will usually be necessary. The reanalysis will be similar but less approximate than the detailed structural evaluation procedure. In most cases, the effects of strengthening and/or stiffening of an existing building will reduce the modal periods of vibration and increase the spectral demand on the building. As Figure 1 illustrates, one or more analysis iterations may be required to reconcile the modified capacity of the building with seismic demand. Each alternative upgrading concept will be checked for compliance with the appropriate performance goal. At a minimum, the design will comply with performance goal D, Substantial Life Safety.

5.3.4.2. The cost effectiveness of upgrading wind and/or seismically deficient existing buildings will be evaluated on the basis of data obtained from the evaluation and report phase, the detailed structural analyses, and the development of retrofit design concepts. The options of taking no action and building replacement will again be considered as well as that of upgrade.

5.3.5. Phase V consists of final design and the preparation of contract documents for construction. The final design will be done on the basis of the results from the detailed structural analysis, development of retrofit design concepts, and the cost benefit analysis. The final design will include a complete analysis of the upgraded structure, completed drawings of all details for the project, and a detailed cost estimate. The final documents will completely describe the basis of design without the need to refer to the previous analysis and development work done during the rapid evaluation and detailed evaluation phases.

5.3.6. Phase VI is the construction phase.

5.4. Evaluation Phase Methodology. In conducting the evaluation phase, it is important to limit the structural analysis performed to that necessary to establish with reasonable sufficiency for programming purposes: (1) that the building justifies mitigation by either upgrade or replacement; and, if by mitigation, (2) the schematic definition of a design concept for the structural upgrade necessary for the building to comply with the performance goals; and (3) a programming cost estimate. These actions will be reassessed to much greater detail and accuracy as part of Phase IV, Preliminary Design, after the programmed project has been approved for construction.

5.4.1. With few exceptions regarding some irregular and essential buildings, the computational work required for evaluation by this ETL can be performed easily with a pocket calculator. When applied by a structural engineer with experience in seismic and/or wind design criteria, the approximate structural analysis methods prescribed should result in sufficient accuracy to establish the requirement for mitigation and to provide a schematic definition of a strengthening concept and cost estimate adequate for programming purposes. In modern professional design firms structural computations are most commonly done with computers. Hence, many evaluators may elect to conduct the simplified analysis by computer analysis for reasons of personal convenience. This approach based on approximate analysis for evaluation is consistent with the advice given in FEMA 178. This guidance document recommends that the analysis prescribed in the handbook be carried out using appropriate simple procedures so that truly hazardous buildings may be quickly identified (and with reasonable economy). With this approach, it is recognized that many buildings will not pass the tests given in FEMA 178. Failure to pass the test does not automatically indicate a hazardous building. For marginal or questionable buildings, a more detailed investigation may be justified. FEMA 178 recommends that such an investigation would be most effective if done as part of an upgrading program.

5.4.2. The evaluation phase includes the steps listed below. Figure 2 illustrates the procedures taken in evaluation when there is only one performance objective: substantial life safety prescribed as the basic requirement of FEMA 178. Attachment 5 describes in more detail the methodology and approximate methods used in conducting the rapid structural evaluation. Attachment 8 provides description of the detailed structural evaluation.

5.4.3. Step 1: Establish Evaluation Data File.

5.4.3.1. The Base Civil Engineer (BCE) can be expected to provide geotechnical reports on site soil conditions, record drawings, description of remodeling work, and in limited cases, specifications and calculations. For “pre-engineered” buildings, the BCE will attempt to obtain and provide detailed shop drawings and analysis from the building manufacturer. Additional useful information may be obtained through interviews with the structural and geotechnical engineers of the DoD agency responsible for major design and construction at the base. This information may include performance data such as earthquake and wind evaluations, and prior earthquake and wind damage. Seismicity and wind evaluation load conditions will be as prescribed later in this ETL. The evaluator will conduct an initial visit to the office of the Base Civil Engineer (BCE). All pertinent data from these documents and interviews will be entered on the evaluation data form provided in Attachment 6. The building site is then visited to confirm the information collected at the BCE office and to gain any additional information relevant to the structural condition of the building, including that required by the evaluation data form. This includes any past modifications and damage that may have been caused by earlier wind, earthquake, or flooding events; lack of maintenance; and insects. FEMA 178 recommends that such an investigation would be most effective if done as part of an upgrading program. Sketches and photographs of significant items related to structural performance should be made.

5.4.4. Step 2: Rapid Structural Evaluation.

5.4.4.1. The rapid structural evaluation, for both wind and seismic vulnerability, is accomplished using the procedures described in Attachment 5. The evaluation statements of FEMA 178, for seismic, and Attachment 7, for wind, are used with the quick check procedures. The rapid structural evaluation procedure is the same as that described in FEMA with two exceptions. First, the seismic lateral force equations which follow are used. Second, in addition to the check of sufficiency as featured in FEMA 178 for the design earthquake (e.g., EQ-II), a check is made of structural sufficiency to resist without collapse the effects of EQ-III, the Collapse Limit Earthquake (CLE). Thus, a two-level procedure (Level A and Level B) is used in conducting the rapid structural evaluation. The quick checks are approximate calculations for stresses and deflections in the basic components of the lateral-force-resisting system. The

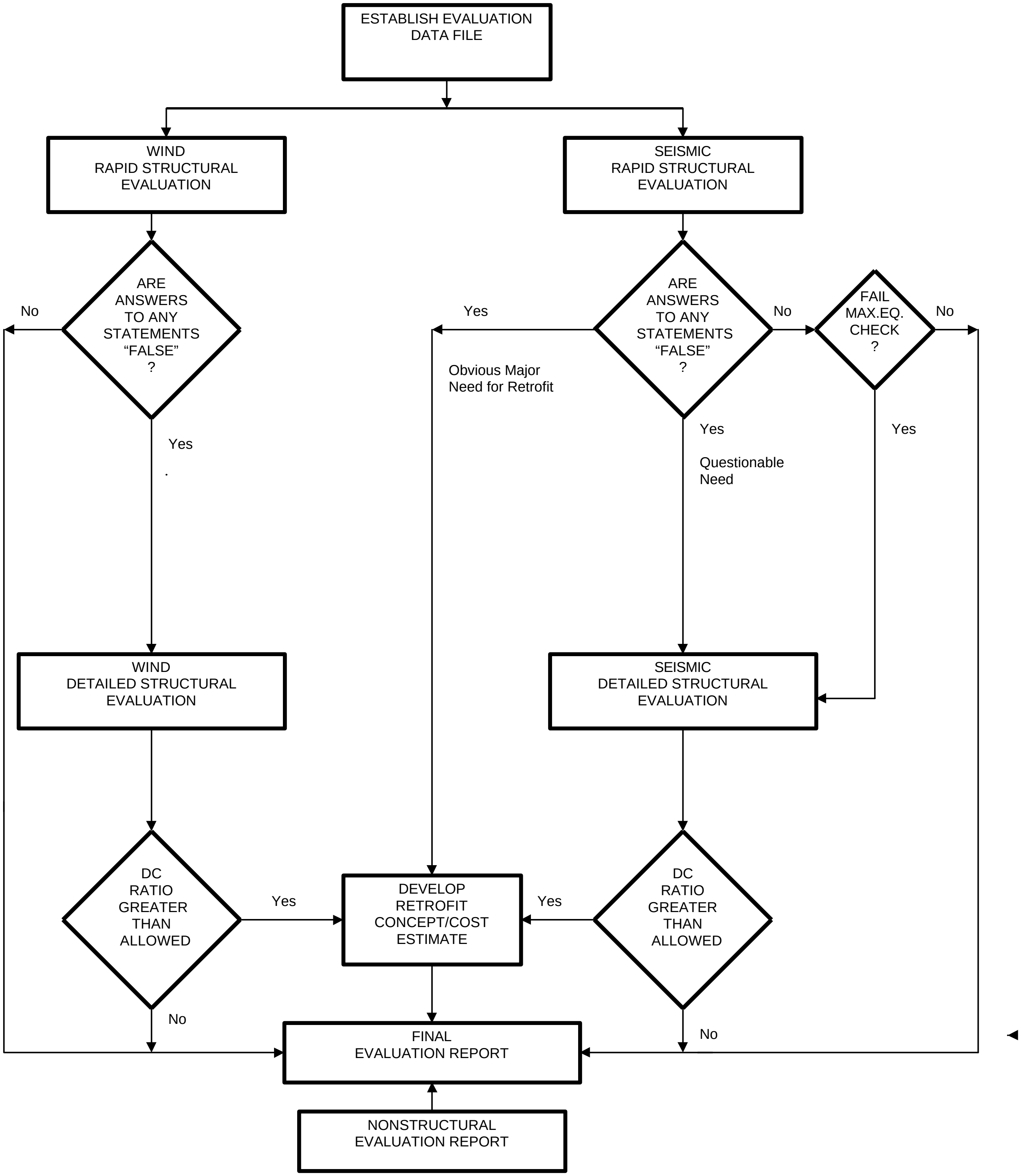

Figure 2. Evaluation and Report Phase Procedure

calculations are analogous to the preliminary calculations often made by engineers as an indicator of basic capacity. The same quick check formula and procedures are used in conducting Level A and Level B rapid evaluations. The results of rapid structural evaluation will be recorded on the Rapid Structural Evaluation Summary provided in Attachment 6.

5.4.4.2. When evaluating performance under seismic loadings, the lateral forces will be computed as discussed in Attachment 3 in accordance with FEMA 178 with the following exceptions:

5.4.4.2.1. For Level A of the rapid evaluation quick check calculations, Equation 2-4 of FEMA 178 is changed to the following equation:

$$C_s = 0.85\frac{S_{adl}}{RT^n} \qquad (1)$$

where:

S_{adl} = $F_v S_{DL}$
= Design spectral acceleration in the long-period range for the design earthquake

S_{DL} = Spectral acceleration in the long-period range for soil profile Type B for the design earthquake ground motion representing EQ-II spectral acceleration S_{DL} as provided in Attachment 2

F_v = Site coefficient in the long-period range given in Table A3.3 of Attachment 3 using S_{DL} = S_{DL}

R = a response modification coefficient from Table A3.6 of Attachment 3

T = the fundamental period of the building (paragraph A3.2.6, Attachment 3)

n = 1.0 for T < 1.0 second and 2/3 for T > 1.0 second

5.4.4.2.2. Equation 2-5 of FEMA 178 places an upper limit on the value of C_S. For Level A of the rapid evaluation quick check calculations, it is changed to the following equation:

$$C_s = 0.85\frac{S_{ads}}{R} \qquad (2)$$

where:

S_{ads} = $F_a\ S_{DS}$

= Design spectral acceleration in the short-period range for the design earthquake

S_{DS} = Spectral acceleration in the short-period range for soil profile Type B for the design earthquake ground motion representing EQ-II spectral acceleration S_{DS} as provided in Attachment 2

F_a = Site coefficient in short period range given in Table A3.4 of Attachment 3 using $S_{DS} = S_{DS}$

R = Response modification coefficient from Table A3.6 of Attachment 3

5.4.4.3. When evaluating performance under Level B rapid evaluation seismic loadings, the lateral forces shall be computed as discussed in Attachment 3 in accordance with FEMA 178 with the following exceptions:

5.4.4.3.1. For Level B of the rapid evaluation quick check calculations, Equation 2-4 of FEMA 178 is changed to the following equation:

$$C_s = 0.57 \frac{S_{avl}}{RT^n} \qquad (3)$$

where:

S_{avl} = $F_v S_{ML}$
= Design spectral acceleration in the long-period range for the design earthquake

S_{ML} = Spectral acceleration in the long-period range for soil profile Type B for the Collapse Limit Earthquake (CLE) ground motion representing EQ-III spectral acceleration S_{ML} as provided in Attachment 2

F_v = Site coefficient in the long-period range given in Table A3.3 of Attachment 3 using $S_{DL} = 2/3\ S_{ML}$

R = Response modification coefficient from Table A3.6 of Attachment 3

T = Fundamental period of the building (paragraph A3.2.6, Attachment 3)

n = 1.0 for $T < 1.0$ second and 2/3 for $T > 1.0$ second

5.4.4.3.2. Equation 2-5 of FEMA 178 places an upper limit on the value of C_S. For Level B of the rapid evaluation quick check calculations, it is changed to the following equation:

$$C_s = 0.57\frac{S_{avs}}{R} \qquad (4)$$

where:

S_{avs} = F_aS_{MS}
= Design spectral acceleration in the short-period range for the design earthquake

S_{MS} = Spectral acceleration in the short-period range for soil profile Type B for the Maximum Considered Earthquake ground motion representing EQ-III spectral acceleration S_{MS} as provided in Attachment 2

F_a = Site coefficient in short period range given in Table A3.4 of Attachment 3 using S_{DL} = 2/3 S_{MS}

R = Response modification coefficient from Table A3.6 of Attachment 3

5.4.4.4. When evaluating performance under wind loadings, the lateral forces used in the quick check procedures will be as prescribed in Attachment 4.

5.4.4.5. The purpose of the rapid structural evaluation is to eliminate from further evaluation buildings with complete lateral-force-resisting systems possessing a minimum strength. Short and quick evaluation tools, such as true-false statements of FEMA 178 and Attachment 7, are used as responses to "Quick-Check" strength and stiffness calculations. The detailed evaluation procedures of Attachment 8 will only be applied after the rapid structural evaluation results have been reported on the Rapid Structural Evaluation Executive Summary, with attachments (Attachment 6), to the approving authority.

5.4.4.6. The building may be found to be obviously safe without further evaluation, obviously not safe without the need for further evaluation, or correctable deficient. If the building is determined to meet the minimum performance objective, the final report is prepared and the building is placed in performance objective Category V on the seismic risk inventory. As stated in ETL 93-3 (Reference 2), Category V includes buildings which are exempt from structural evaluation and require only further evaluation for nonstructural hazards. When structural mitigation is required, the report shall be prepared and submitted with a recommendation to either proceed with a detailed structural evaluation or to program the building for the appropriate mitigation.

5.4.5. Step 3: Detailed Structural Evaluation.

5.4.5.1. Detailed structural evaluation is required when a rapid structural evaluation can not be easily used to conclude that a building meets the specified performance objectives. The detailed structural evaluation is done, considering all applicable performance requirements using the procedures of Attachment 8, after the rapid structural evaluation is completed. In the detailed analysis, specific structural elements are examined, using analytical methods to establish if there is sufficient strength and ductility available to support the imposed loads. A detailed structural analysis is warranted if the evaluation could produce evidence that would allow a building which failed the rapid structural evaluation to pass the structural evaluation. Some of the methods which can be used to do a detailed structural evaluation are explained in Attachment 9.

5.4.5.2. A standard building code lateral force analysis is done for the detailed evaluation for wind using approximate analysis methods and load determinations as discussed in Attachments 4 and 8.

5.4.5.3. To determine the adequacy of seismic resistance, the elastic analysis method of post-yield analysis is used as provided in Attachment 8. This methodology is Method 1 of two post-yield analysis methods as provided in AFJMAN 32-1049V3, *Seismic Design Guidelines for Upgrading Existing Buildings* (Reference 11) and simplified with approximate analysis used to calculate force, stress, and drift demands. In the event any of the conditions in A8.13.4.6 exist, the building must be analyzed instead in accordance with Method 2, the capacity spectrum method, which is summarized in Attachment 8 and described in detail in Reference 11. These requirements do not prohibit the use of other properly substantiated inelastic response spectrum methods or inelastic time-history procedures. Attachment 9 provides some approximate analysis methods that can be used with a hand calculator to solve evaluations under this ETL. The spectral accelerations of either EQ-I, EQ-II or EQ-III are used, depending upon the particular performance objective.

5.4.5.4. Figure 3 is a flow chart for the seismic detailed structural evaluation procedure as it includes several performance goals involving EQ-I, EQ-II, and EQ-III. As shown, compliance of the building response with relevant performance goal C criteria (C1, C2, and C3) involving EQ-II is assessed first. If the demand/capacity ratios (DCR) are greater than the allowables for the particular performance goal, the project is placed under consideration for retrofit. If the DCR values do not exceed the allowables and if Performance goal B also applies, a detailed structural evaluation will be conducted for EQ-I forces. As shown in Table 1, the EQ-I earthquake represents performance goal B. According to the performance criteria of Attachment 8, the member strength (MS) of columns and girders is checked for compliance at each floor level. If the building fails in performance, it is eliminated from further evaluation and placed under consideration for retrofit. If its performance is satisfactory under the EQ-I forces and its NDFEF exceeds 1.5, the building is placed under evaluation considering EQ-III and performance goal D. After this point, detailed structural evaluation for the multi-level performance criteria building is completed. If the building fails the DCR check, the seismic and any wind deficiencies are jointly considered and a schematic mitigation project and first order cost estimate are developed for programming purposes.

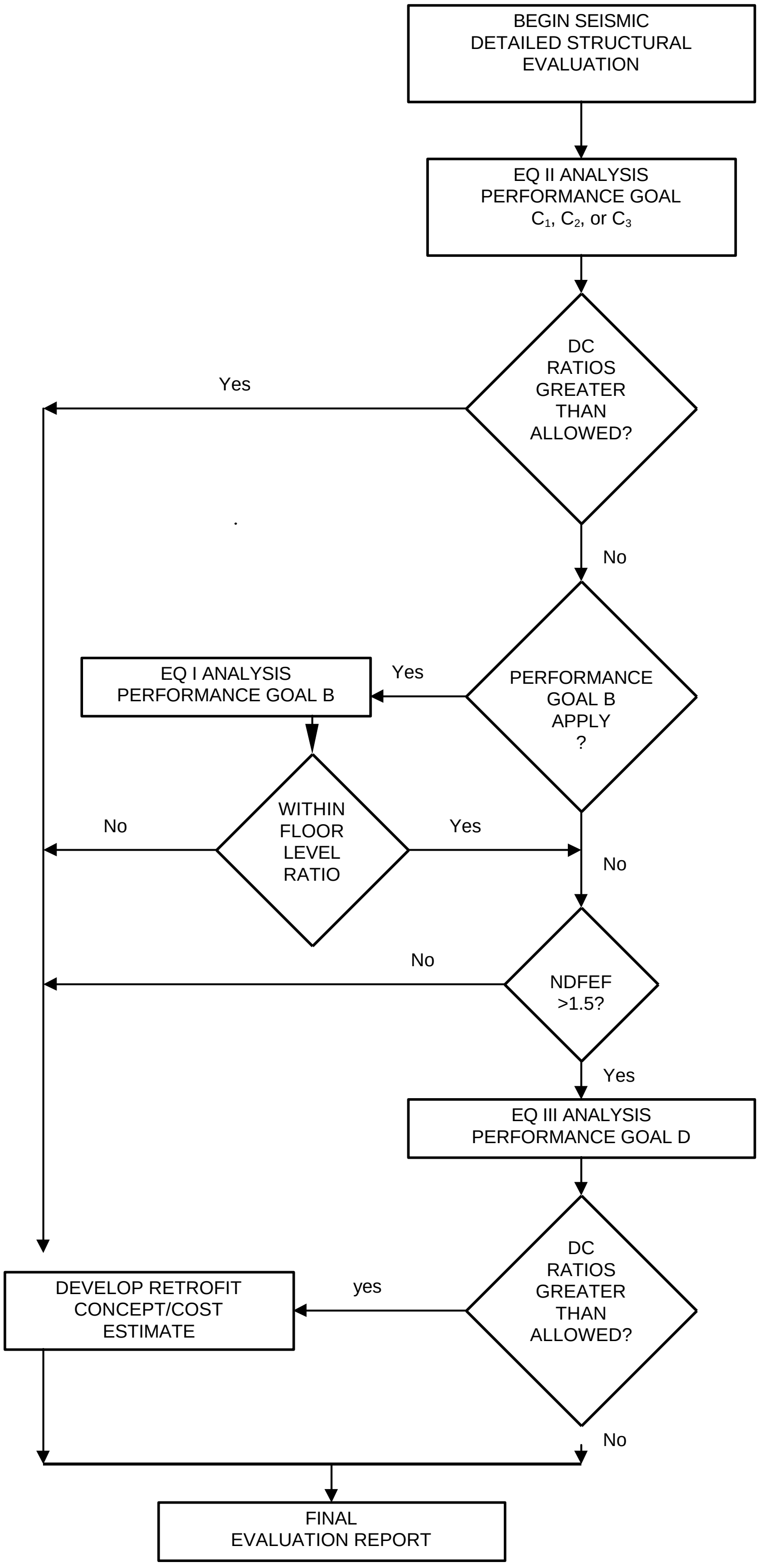

Figure 3. Seismic Detailed Structural Evaluation Procedures

5.4.5.5. When conducting a detailed structural evaluation under seismic loadings computed as discussed in Attachment 3, the seismic coefficient, C_S, will be computed using the following equation:

$$C_s = 0.85 \frac{S_{adl}}{T^n} \qquad (5)$$

where:

S_{adl} = $F_v S_{AL}$
= Spectral acceleration in the long-period range for the Level II Detailed Evaluation earthquake

S_{AL} = the EQ-I, EQ-II or EQ-III spectral acceleration at 1.0 period for soil profile Type B from Attachment 2

F_v = Site coefficient in long-period range for the Level II Detailed Evaluation earthquake given in Table A3.3 of Attachment 3

5.4.5.6. The value of C_S need not be greater than the following limiting value:

$$C_s = 0.85 S_{ads} \qquad (6)$$

where:

S_{ads} = $F_a S_{AS}$

S_{AS} = the EQ-I, EQ-II or EQ-III spectral acceleration at 0.2 period for soil profile Type B from Attachment 2

F_a = Site coefficient in long-period range for the Level II Detailed Evaluation earthquake given in Table A3.4 of Attachment 3

5.4.6. Step 4: Develop Retrofit Concept and Cost Estimate. When a building fails the rapid structural evaluation and/or the detailed structural evaluation, a mitigation mechanism must be recommended. Mitigation may include occupancy reduction, abandonment, demolition, replacement, or rehabilitation or upgrade. Specific threat will be cited as justification for occupancy reduction or abandonment. Economic justification will be provided when demolition or replacement is recommended. A schematic strengthening concept will be developed, to include program level estimates for rehabilitation or upgrade recommendations.

5.4.7. Step 5: Final Evaluation Report. An evaluation report will be prepared using the evaluation report formats provided in Attachment 6. The report will include, as minimum, the (1) applicable evaluation executive summary; (2) evaluation data forms for wind or seismic (both where applicable); (3) the evaluation results form; (4) complete evaluation statements from FEMA 178 or Attachment 7; (5) a recommended mitigation mechanism when deficiencies are identified; (6) calculations and findings; and (7) an engineer's statement as to the level of confidence in the available building information and the applicability of the methodology to the building evaluated. Other attachments may be provided by the Engineer of Record as deemed necessary.

5.5. Qualifications of Evaluator, Designer, and Reviewer for Application of Methodology for Evaluation of Existing Buildings. All evaluation work, mitigation recommendations, and the design of rehabilitation work will be prepared under the direct supervision of a professionally registered practicing structural engineer with recent experience in the design of buildings to support the loads imposed by either earthquake or high wind events.

6. Point of Contact: Mr. James L. Lafrenz, P.E., HQ AFCESA/CESC, DSN 523-6332, commercial (850) 283-6332, or INTERNET lafrenzj@afcesa.af.mil.

William G. Schauz, Colonel, USAF
Director of Technical Support

9 Atch
1. References
2. Ground Motion Criteria
3. Seismic Effects and Force Demands
4. Wind Effects and Force Demands
5. Rapid Seismic and Wind Structural Evaluation
6. Evaluation Data Collection and Final Evaluation Report Forms
7. Wind Evaluation Statements

7a. General Sets of Wind Evaluation Statements

7b. Wind Evaluation Statements for 15 Common Building Types

8. Detailed Structural Evaluation
9. Distribution List

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Ground Motion Criteria

A2.1. New USGS Ground Motion Hazard Maps. Significant additional earthquake data, understanding, and other advances have been obtained in the last 20 years, which make the A_a and A_v ground motion hazard maps out of date. In June 1996, the U.S. Geological Survey (USGS) released a new generation of probabilistic ground motion hazard maps based on this new understanding and an extensive national review process that featured a series of regional workshops on the initial version of these new maps. These new maps represent a significant upgrade of seismic ground motion hazard nationwide. They allow direct definition of the design spectra by mapping the response spectral ordinates at different periods. The early maps were included in the initial draft of this ETL. The finalized new USGS ground motion hazard maps are used in this ETL. Paragraph A2.7 provides the means to adjust the mapped response acceleration parameters for other probabilities of exceedance required in conducting evaluation in accordance with this ETL.

A2.2. Related Ground Motion Hazard Maps of Emerging NEHRP Provisions. With some modifications by the Building Seismic Safety Council (BSSC) imposing maximum and minimum values for certain regions of the United States, some of the maps are intended to be included in the 1997 Edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings (*1997 *NEHRP Provisions*) and FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (Reference 13). The principal limitation is the maximum limit established for regions of high seismicity within 10 kilometers (6.2 miles) of known fault sources with short return periods, essentially certain areas of coastal California. In FEMA 273 and the 1997 *NEHRP Provisions*, the earthquake described by the modified maps for ground shaking with a 2% probability of exceedance in 50 years (2%/50 years) is termed the Maximum Considered Earthquake (MCE).

A2.3. Use of Single Exposure Period of 50 Years. In conventional building code provisions like those of AFJMAN 32-1049, the “design earthquake” ground motions are based on an estimated 90 percent probability of not being exceeded in a 50-year period (about a 500-year return period). The evaluation criteria of this ETL use several return periods (70, 500, 1000, 2500, and 4000 years). These ground motions can be expressed using the notation of 90 percent probability of not being exceeded in a certain exposure time period. For example, the notation 90 percent probability of not being exceeded in the exposure periods of 100 and 250 years can be used to calculate the return periods of 1000 and 2500 years, respectively. However, consistent with the development of the 1997 *NEHRP Provisions*, the single exposure time of 50 years is used in this ETL which is generally taken as the useful life of a building without rehabilitation. Thus, different levels of probability or return period are expressed as percentage probability of exceedance in 50 years. Specifically, 10 percent probability of exceedance in 50 years is a return period of about 500 years, 5 percent probability of exceedance in 50 years is a return period of about 1000 years, and 2 percent probability of exceedance in 50 years is a return period of about 2500 years.

A2.4. The Collapse Limit Earthquake (CLE). The new generation USGS products include hazard maps for ground shaking with a 10% chance of exceedance in 50 years (10%/50), a 5% chance of exceedance in 50 years (5%/50), and a 2% chance of exceedance in 50 years (2%/50). These probabilities correspond to shaking that is expected to occur, on the average, about once every 500 years (or more exactly, 474 years), 1000 years, and 2500 years, respectively. As stated above, the 10%/50 year maps represent the conventional "design earthquake" as usually mapped in building code provisions. As discussed in paragraph 5.2.1, the focus of building code design (as well as the ICSSC Standards and FEMA 178) is on life-safety and non-collapse in the event of the rare, but possible, large earthquake. The 2%/50 year (2500 year return period) maps represent this level earthquake, hereafter referred to as the Collapse Limit Earthquake (CLE) for non-essential buildings. In the case of essential buildings, the earthquake ground motion with a 4000 year return period is used as the Collapse Limit Earthquake (CLE). Thus, in this ETL, the Collapse Limit Earthquake (CLE) represents EQ-III of either a 4000 year or 2500 year return period, depending on whether the occupancy categories are I (Immediate Occupancy) or III (High Risk) and IV (Other Buildings), respectively.

A2.5. Seismic Hazard Maps Used in Evaluation. The USGS maps produced for the two earthquake levels, EQ-II and EQ-III, are provided in this ETL; the 10%/50 year maps are provided in Figures A2.1 and A2.2 and the 2%/50 year maps are provided in Figures A2.3 and A2.4. The ground motion spectral accelerations for the 70, 1000, and 4000 return year events can be determined using the procedure described in paragraph A2.7 and illustrated in Figure A2.7. The hazard maps for each return period are the spectral response acceleration maps (as a percent of gravity) at periods of 0.2 sec and 1.0 sec for a Type B soil profile. A Type B soil profile is defined in the 1994 *NEHRP Provisions* (Reference 8) as "Rock with 760 m/s < $v_s \leq$ 1500 m/s (2,500 ft/sec < $v_s \leq$ 5,000 ft/sec)." More spectral ordinates could have been calculated, but this would have unjustifiably increased the number of maps. The study (Reference 16) has shown that the two ordinates as mapped provide a reasonable approximation to a spectrum determined using all available ordinates. All maps are for 5 percent damped systems. Maps for base level rock (soft) motion are included in Figures A2.5 and A2.6 to permit construction of spectral acceleration where site specific determinations are necessary.

A2.6. Determination of Mapped Acceleration Parameters. There are multiple performance objectives and five different return levels specified for use in the evaluation of buildings at Air Force installations. The five levels have return periods of 70, 500, 1000, 2000, and 4000 years. Spectral accelerations for the 500 and 2500 year return periods are provided in the maps of Figures A2.1 through A2.4. Spectral accelerations for each of the other three earthquakes can be determined with graphical or analytical methods using the hazard maps for return periods of 500 and 2500 years.

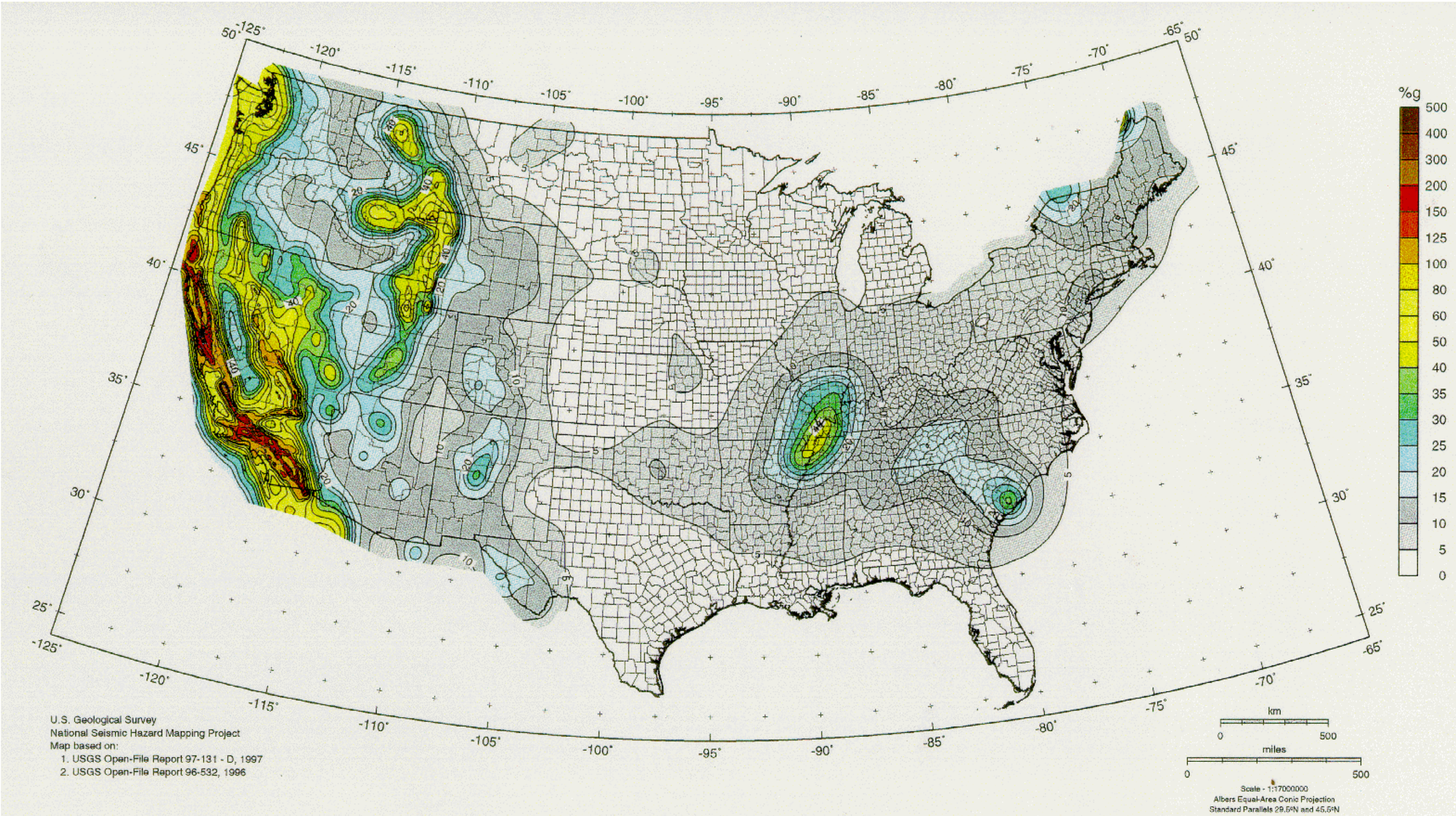


Figure A2.1. **Horizontal Spectral Response Acceleration (%g) for 0.2 Sec Period (5% of Critical Damping) With 10% Probability of Exceedance in 50 Years Firm Rock - 760 m/sec Shear Wave Velocity**

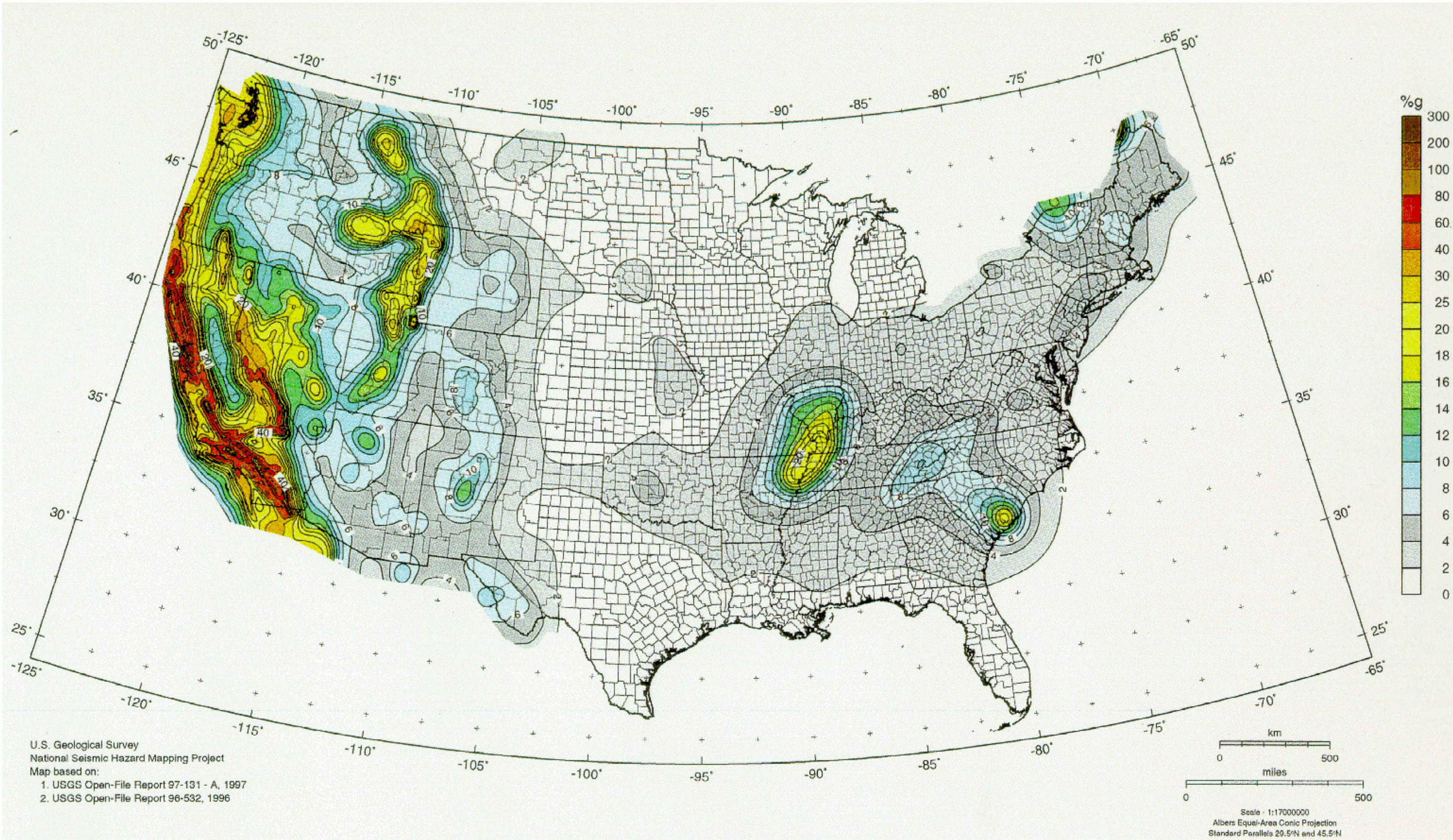


Figure A2.2. **Horizontal Ground Acceleration (%g) (5% of Critical Damping)
With 10% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec Shear Wave Velocity**

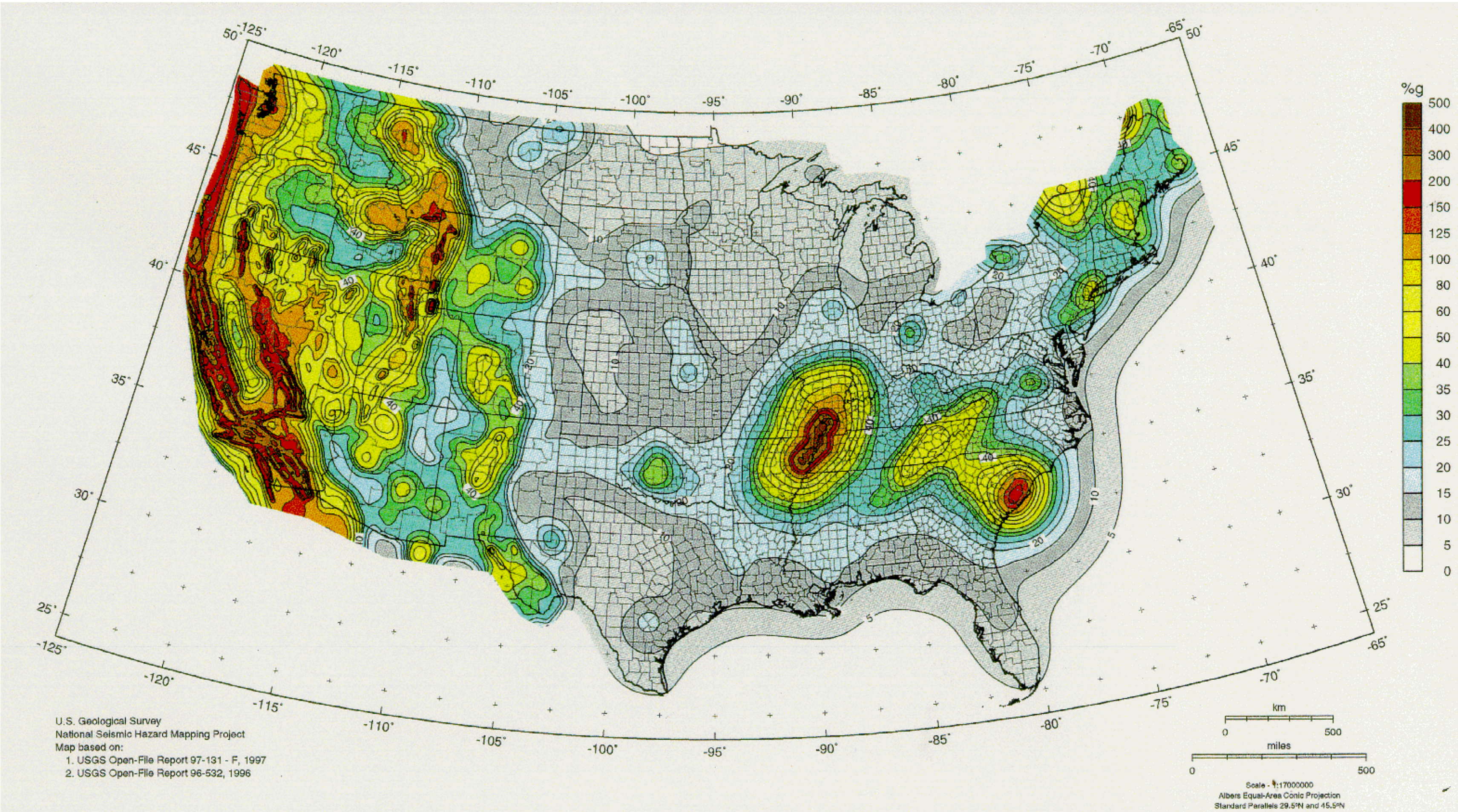


Figure A2.3. **Horizontal Spectral Response Acceleration (%g) for 0.2 Sec Period (5% of Critical Damping)**
With 2% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec Shear Wave Velocity

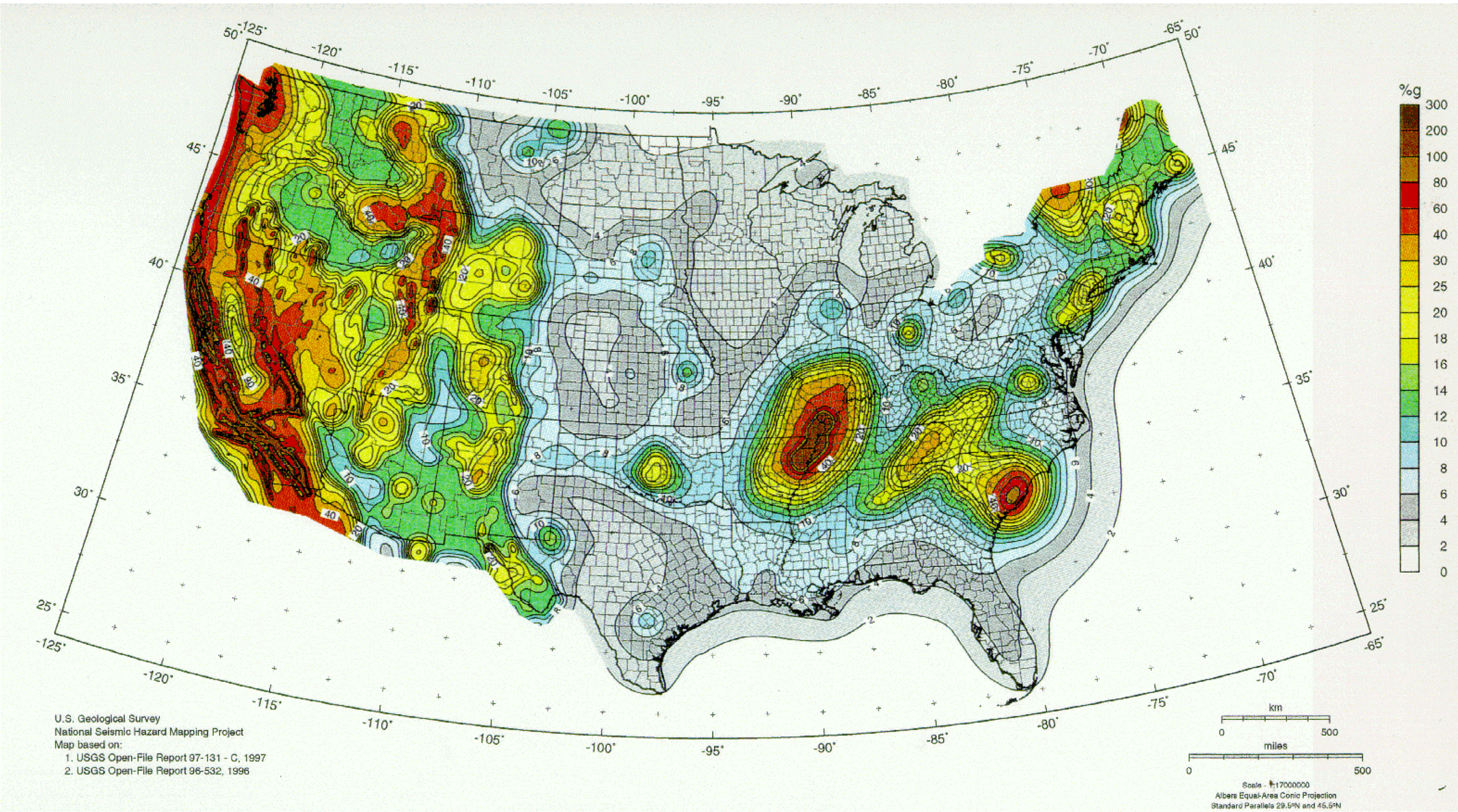


Figure A2.4. **Horizontal Ground Acceleration (%g) (5% of Critical Damping)**
With 2% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec Shear Wave Velocity

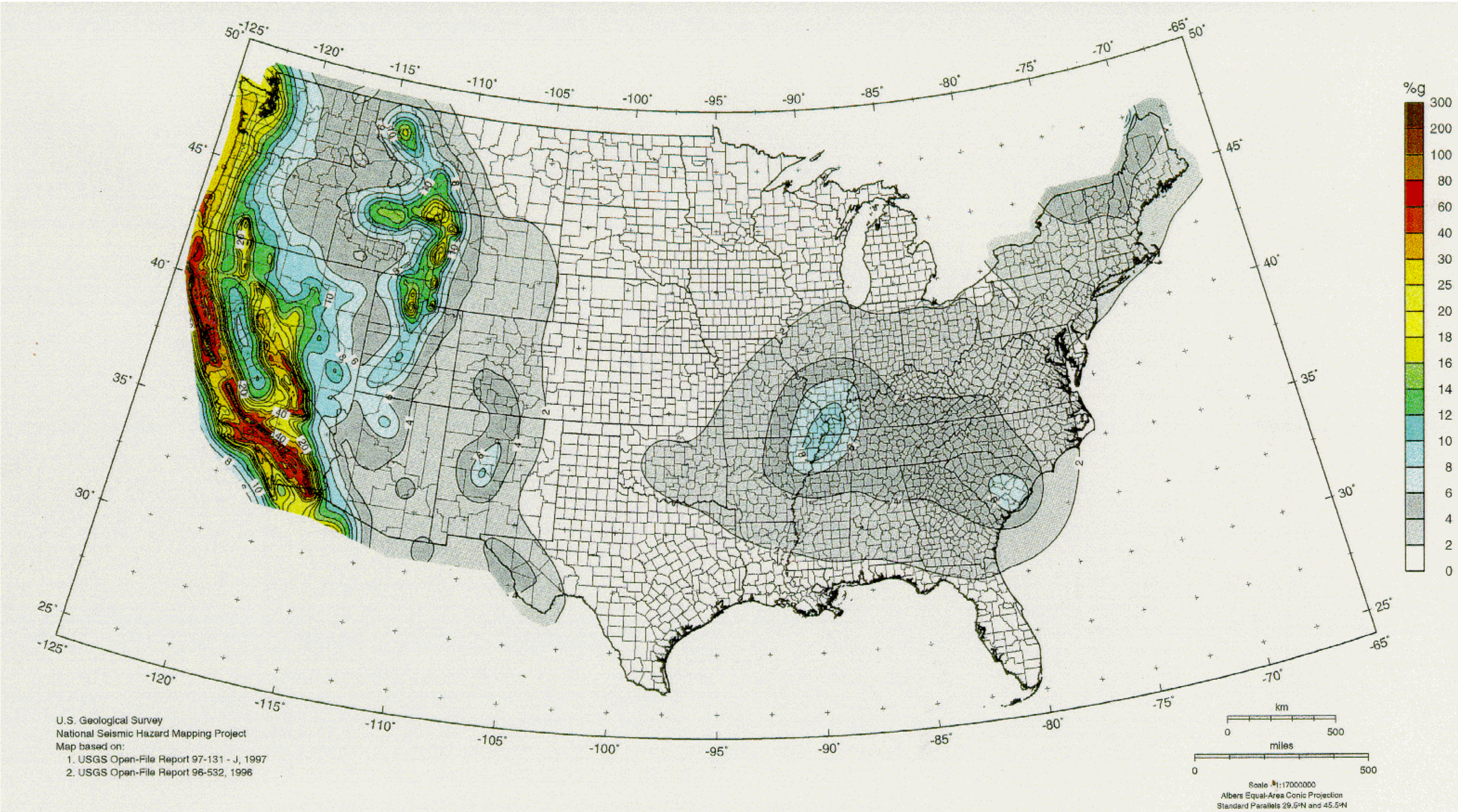


Figure A2.5. **Horizontal Spectral Response Acceleration (%g) for 1.0 Sec Period (5% of Critical Damping) With 10% Probability of Exceedance in 50 Years Firm Rock - 760 m/sec Shear Wave Velocity**

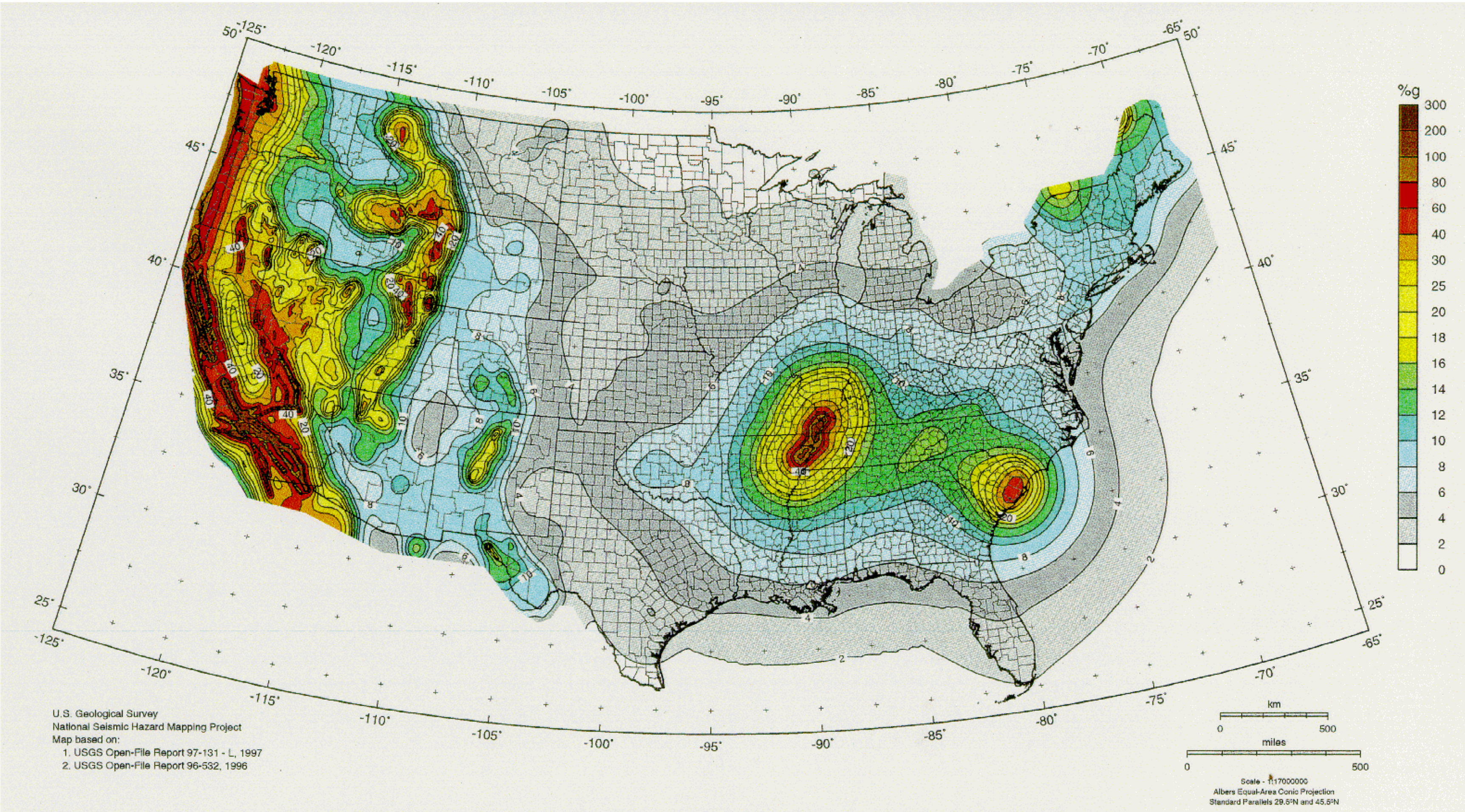


Figure A2.6. **Horizontal Spectral Response Acceleration (%g) for 1.0 Sec Period (5% of Critical Damping) With 2% Probability of Exceedance in 50 Years Firm Rock - 760 m/sec Shear Wave Velocity**

A2.6.1. Analytically, the other required mapped acceleration parameters may be determined using the equations and procedures based on those provided in paragraph 2.6.1.3 of FEMA 273 (Reference 13).

$$\ln(S) = \ln(S_{10/50}) + [\ln(S_{2/50}) - \ln(S_{10/50})][0.606 \ln(P_R) - 3.73]$$

where:

$\ln(S)$	=	the natural logarithm of the spectral acceleration parameter (short period or 1 second period) at the desired probability of exceedance
$\ln(S_{10/50})$	=	the natural logarithm of the spectral acceleration parameter (short period or 1 second period) at a 10%/50 year exceedance rate
$\ln(S_{2/50})$	=	the natural logarithm of the spectral acceleration parameter (short period or 1 second period) at a 2%/50 year exceedance rate
$\ln(P_R)$	=	the natural logarithm of the return period corresponding to the exceedance probability of the desired hazard level

and the return period P_R at the desired exceedance probability may be calculated from the equation:

$$P_R = \frac{1}{1 - e^{0.02 \ln(1 - P_{E50})}}$$

where P_{E50} is the probability of exceedance in 50 years of the desired hazard level.

A2.6.2. The graphical method is represented in Figure A2.7. As indicated, it is to plot the spectral accelerations from the two given maps on log-log paper, draw a straight line through the two known data points, and then determine the required mapped response acceleration parameters for other probabilities of exceedance by interpolation or extrapolation, as required.

A2.6.3. Spectral accelerations for each of the five earthquakes are provided for major continental locations in Table A2.1. The National Design Force Exceedance Factor (NDFEF) is calculated as the ratio of short-period spectral accelerations of the 2500 (EQ-III) and 50 year (EQ-II) events. The NDFEF value for each installation is also listed. The EQ-III will be used to evaluate buildings for life-safety where the NDFEF exceeds a factor of 1.5. The evaluation of those “benchmark buildings,” exempted from evaluation by ETL 93-3 (Reference 2), which have a performance category of Immediate Occupancy or High Risk, is required when the NDFEF exceeds 1.5. In order to facilitate site specific assessments, peak ground accelerations are listed in Table A2.2.

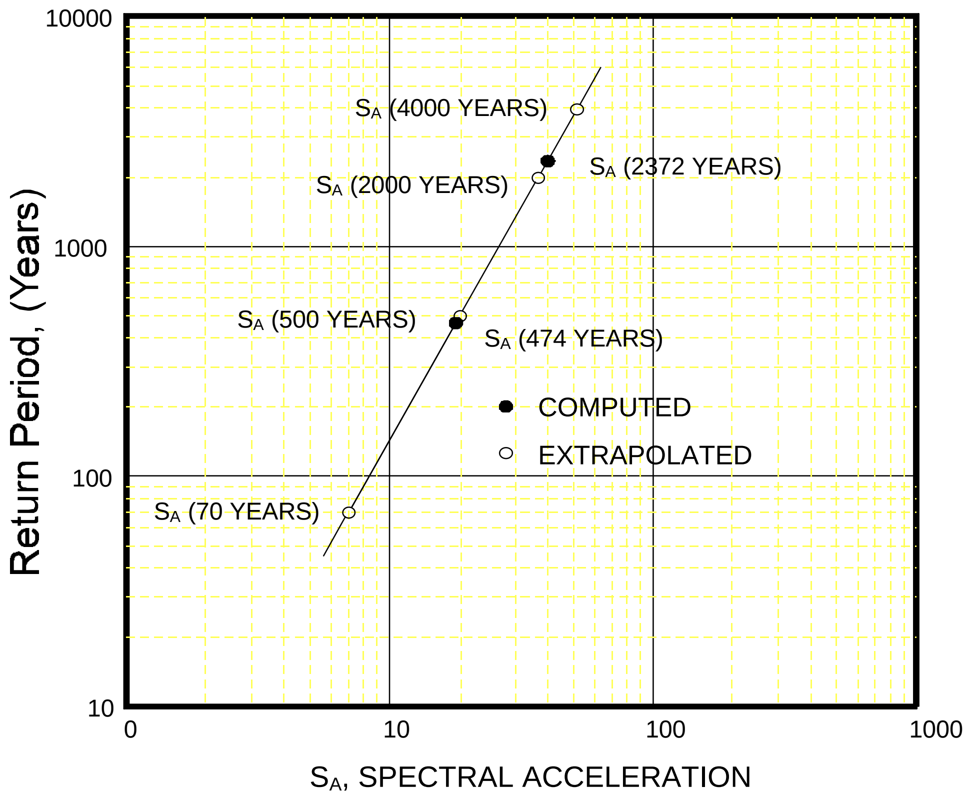


Figure A2.7. Graphical Determination of Earthquakes of Other Return Periods

A2.7. Other Determinations of Ground Shaking Hazards. In addition to the basic procedures described above using the USGS probabilistic maps, ground shaking hazards may be determined by probabilistic site specific procedures as approved by the Major Command Civil Engineer. As stated in Tables A3.3 and A3.4, site specific procedures will be used when the building is located on (1) Type F soils and (2) Type E soils when the spectral acceleration S_{DL} equals or exceeds 0.5g. Site specific procedures would also be used should a time history response analysis of the building be performed as part of the evaluation.

A2.8. High Seismicity Regions Within 10 Kilometers of Major, Active Faults. As earlier indicated, the basic and site specific probabilistic procedures may not yield appropriate estimates of the ground shaking for sites located within about 10 kilometers of major, active faults. FEMA 273 advises that for faults with high activity rates, these procedures may overestimate the ground motion for probabilities of exceedance greater than about 5% in 50 years. Accordingly, for building sites located in high seismicity regions within 10 kilometers of major, active faults, the ground shaking hazards used in evaluation shall be established using the Maximum Considered Earthquake (MCE) maps provided in FEMA 273.

A2.9. Spectral Accelerations for Hawaii, Alaska, and Overseas Locations. The new U.S. Geological Survey probabilistic ground motion maps have been produced for the contiguous states only. In applying the methodology of this ETL in Alaska and

Hawaii at other than for Eielson, Elmendorf, and Hickam Air Force Bases, the Maximum Considered Earthquake (MCE) maps of FEMA 273 and other NEHRP maps may be used in determining the required spectral acceleration values. The information for these three bases is included with that for bases within the contiguous 48 states in Tables A2.1 and A2.2. Development of other spectral acceleration values outside of the contiguous 48 United States shall be approved by the Air Force Major Command Civil Engineer.

A2.10. Seismic Force Demands. The associated response spectra for evaluation are provided in Attachment 3.

Table A2.1. Spectral Accelerations for Evaluation (%g)

INSTALLATION	SHORT-PERIOD RANGE						LONG-PERIOD RANGE				
	RETURN PERIOD (YRS)						RETURN PERIOD (YRS)				
	70	500	1000	2500	4000	NDFEF	70	500	1000	2500	4000
Alabama											
Maxwell/ Gunter Annex	1.6	5.8	9.0	16.2	21.9	2.8	0.7	2.8	4.5	8.3	11.4
Alaska											
Eielson	20.0	53.0	78.0	123.0	158.0	2.3	8.0	21.0	31.0	49.0	63.0
Elmendorf	45.0	100.0	137.0	200.0	246.0	2.0	18.0	40.0	55.0	80.0	98.0
Arizona											
Luke	4.5	11.2	15.4	23.5	29.1	2.1	1.4	3.3	4.5	6.8	8.3
Davis Monthan	4.2	13.0	19.4	32.9	43.1	2.5	1.2	3.6	5.3	9.0	11.7
Arkansas											
Little Rock	3.0	15.4	27.2	58.1	85.6	3.8	0.7	4.4	8.6	20.6	32.3
California											
Beale	14.2	26.2	32.6	43.3	50.1	1.7	6.1	11.4	14.2	19.1	22.2
Edwards	22.4	49.1	64.8	93.3	112.6	1.9	12.1	24.3	31.0	42.9	50.7
March	80.3	125.5	146.9	180.8	201.2	1.4	27.9	48.7	59.3	76.8	87.8
McCellan	15.7	28.2	34.8	45.7	52.6	1.6	7.4	13.0	15.8	20.5	23.4
Los Angeles	65.4	113.5	137.8	178.1	203.2	1.6	18.7	38.4	49.5	69.2	82.2
Travis	64.1	117.3	145.1	192.3	222.1	1.6	19.6	41.2	53.6	75.8	90.5
Vandenberg	25.6	60.2	81.3	121.0	148.4	2.0	9.1	21.3	28.8	42.9	52.6
Colorado											
Buckley	2.3	7.4	11.2	19.2	25.3	2.6	0.7	2.3	3.4	5.9	7.8
Falcon	2.1	6.9	10.5	18.4	24.4	2.7	0.7	2.2	3.3	5.7	7.5
Peterson	2.3	7.2	10.8	18.2	23.9	2.5	0.8	2.4	3.5	6.0	7.8
USAF Academy	2.5	7.5	11.1	18.5	24.1	2.5	0.7	2.4	3.6	6.2	8.2

Table A2.1. Spectral Accelerations for Evaluation (%g) (Continued)

INSTALLATION	SHORT-PERIOD RANGE						LONG-PERIOD RANGE				
	RETURN PERIOD (YRS)						RETURN PERIOD (YRS)				
	70	500	1000	2500	4000	NDFEF	70	500	1000	2500	4000
Connecticut											
Delaware											
Dover	1.8	6.8	10.9	20.0	27.3	2.9	0.6	2.2	3.5	6.4	8.8
Florida											
Eglin	0.8	3.3	5.5	10.5	14.7	3.2	0.4	1.8	3.0	5.8	8.2
Hurlburt Field	0.8	3.2	5.2	10.0	13.9	3.1	0.4	1.8	2.9	5.7	8.1
Homestead	0.1	1.1	2.2	6.0	10.0	5.7	0.1	0.4	0.9	2.3	3.8
MacDill	0.4	2.3	4.1	8.8	13.0	3.8	0.2	1.0	1.8	3.9	5.7
Patrick	0.5	2.5	4.5	9.8	14.6	3.9	0.2	1.0	1.9	4.3	6.5
Tyndall	0.7	2.9	4.7	8.9	12.4	3.1	0.4	1.7	2.8	5.4	7.6
Georgia											
Dobbins	3.9	11.5	17.0	28.3	36.8	2.5	1.3	4.3	6.7	11.9	15.9
Robins	2.5	8.1	12.1	20.8	27.5	2.6	1.2	3.8	5.8	9.9	13.1
Moody	1.5	5.2	8.1	14.6	19.7	2.8	0.7	2.7	4.3	8.0	11.0
Hawaii											
Hickam	10.0	22.0	30.0	45.0	56.0	2.0	4.0	9.0	12.0	18.0	22.0
Idaho											
Mountain Home	7.2	18.8	26.3	41.0	51.6	2.2	2.3	5.7	8.0	12.3	15.4
Illinois											
Scott	7.1	24.0	36.8	64.7	86.4	2.7	1.4	6.1	10.3	20.5	29.1
Indiana											
Grissom	1.3	5.0	8.0	14.8	20.4	3.0	0.6	2.5	4.0	7.7	10.7
Iowa											

Table A2.1. Spectral Accelerations for Evaluation (%g) (Continued)

INSTALLATION	SHORT-PERIOD RANGE RETURN PERIOD (YRS)						LONG-PERIOD RANGE RETURN PERIOD (YRS)				
	70	500	1000	2500	4000	NDFEF	70	500	1000	2500	4000
Kansas											
McConnell	1.3	5.0	7.9	14.5	19.8	2.9	0.4	1.8	3.0	5.9	8.4
Kentucky											
Louisiana											
Barksdale	1.3	5.5	9.0	17.4	24.4	3.2	0.2	1.6	3.3	8.7	14.2
Maine											
Maryland											
Andrews	1.8	6.4	10.1	18.3	24.9	2.9	0.6	2.3	3.6	6.5	8.9
Massachusetts											
Hanscom	3.1	11.7	18.7	34.7	47.7	3.0	0.9	3.2	5.1	9.4	12.9
Otis	2.0	8.1	13.4	26.0	36.5	3.2	0.6	2.4	3.9	7.3	10.0
Michigan											
Selfridge	1.2	4.2	6.6	11.8	15.9	2.8	0.4	1.6	2.4	4.3	5.9
Minnesota											
Mississippi											
Columbus	2.2	8.7	14.1	26.8	37.1	3.1	0.9	3.9	6.5	13.1	18.8
Missouri											
Whiteman	1.6	5.2	7.8	13.3	17.5	2.6	0.6	2.5	4.1	8.0	11.2
Montana											
Malmstrom	5.6	12.5	16.6	24.1	29.2	1.9	1.8	4.0	5.3	7.7	9.3
Nebraska											
Offutt	1.0	4.1	6.6	12.7	17.7	3.1	0.3	1.4	2.3	4.5	6.3
Nevada											
Nellis	10.7	30.1	43.4	70.4	90.2	2.3	3.2	8.8	12.7	20.6	26.3

Table A2.1. Spectral Accelerations for Evaluation (%g) (Continued)

INSTALLATION	SHORT-PERIOD RANGE						LONG-PERIOD RANGE				
	RETURN PERIOD (YRS)						RETURN PERIOD (YRS)				
	70	500	1000	2500	4000	NDFEF	70	500	1000	2500	4000
New Hampshire											
New Jersey											
McGuire	2.5	10.3	17.0	32.8	46.1	3.2	0.6	2.6	4.3	8.3	11.7
New Mexico											
Cannon	0.9	3.7	6.1	11.7	16.4	3.1	0.3	1.1	1.9	3.7	5.3
Holloman	3.7	12.9	20.1	35.9	48.3	2.8	1.2	3.8	5.8	9.9	13.1
Kirtland	10.1	27.8	39.7	63.6	81.0	2.3	2.5	7.5	11.2	18.9	24.7
New York											
Griffiss	2.9	9.6	14.7	25.6	34.1	2.7	1.0	3.3	5.1	9.0	12.0
North Carolina											
Seymour Johnson	1.8	7.2	11.7	22.4	31.3	3.1	0.7	3.1	5.2	10.4	14.8
Pope	2.2	9.1	15.1	29.2	41.0	3.2	1.0	4.1	6.7	13.2	18.6
North Dakota											
Grand Forks	0.3	1.5	2.6	5.4	8.0	3.7	0.1	0.4	0.8	1.7	2.5
Minot	0.6	2.3	3.7	6.9	9.6	3.0	0.4	1.0	1.4	2.0	2.5
Ohio											
Rickenbacker	2.1	6.6	9.9	16.9	22.3	2.6	0.8	2.7	4.1	7.0	9.3
Wright-Patterson	2.2	8.1	12.7	23.2	31.5	2.9	0.8	2.9	4.6	8.3	11.2
Oklahoma											
Altus	1.2	5.7	9.9	20.5	29.8	3.6	0.4	1.9	3.2	6.6	9.5
Tinker	2.4	10.5	17.8	35.6	50.8	3.4	0.6	2.6	4.4	9.0	12.8
Vance	1.8	7.4	12.0	23.0	32.1	3.1	0.5	2.2	3.6	7.0	9.8

Table A2.1. Spectral Accelerations for Evaluation (%g) (Continued)

INSTALLATION	SHORT-PERIOD RANGE						LONG-PERIOD RANGE				
	RETURN PERIOD (YRS)						RETURN PERIOD (YRS)				
	70	500	1000	2500	4000	NDFEF	70	500	1000	2500	4000
<u>Oregon</u>											
<u>Pennsylvania</u>											
<u>Rhode Island</u>											
<u>South Carolina</u>											
Charleston	5.6	35.8	68.6	162.3	252.5	4.5	0.9	7.9	16.9	46.0	76.7
McEntire	6.0	22.8	36.6	68.3	94.1	3.0	1.5	6.5	10.9	21.9	31.2
Shaw	4.7	22.3	38.5	79.2	114.7	3.5	1.3	6.5	11.3	23.8	34.8
<u>South Dakota</u>											
Ellsworth	1.6	5.4	8.3	14.9	20.0	2.8	0.5	1.7	2.5	4.2	5.6
<u>Tennessee</u>											
Arnold	4.1	12.4	18.3	30.7	40.0	2.5	1.4	4.9	7.6	13.5	18.3
<u>Texas</u>											
Brooks	0.5	3.0	5.8	13.3	20.5	4.4	0.3	1.0	1.7	3.1	4.3
Carswell	1.0	3.8	6.2	11.6	16.0	3.0	0.4	1.7	2.8	5.7	8.2
Dyess	0.7	2.8	4.5	8.3	11.4	3.0	0.3	1.2	2.1	4.1	5.7
Goodfellow	0.7	2.6	4.2	8.0	11.1	3.1	0.3	1.0	1.6	2.9	3.9
Kelly	0.5	2.9	5.5	12.8	19.7	4.4	0.3	1.0	1.6	3.0	4.1
Lackland	0.5	2.9	5.5	12.8	19.7	4.4	0.3	1.0	1.6	3.0	4.1
Laughlin	0.7	2.4	3.7	6.8	9.3	2.9	0.4	1.0	1.5	2.4	3.0
Randolph	0.6	3.1	5.8	12.9	19.4	4.1	0.2	1.0	1.9	4.0	5.9
Sheppard	1.3	5.4	9.1	17.9	25.4	3.3	0.4	1.8	3.1	6.5	9.4
<u>Utah</u>											
Hill	17.9	52.1	76.0	125.1	161.6	2.4	5.1	17.8	27.6	49.2	66.2

Table A2.1. Spectral Accelerations for Evaluation (%g) (Continued)

INSTALLATION	SHORT-PERIOD RANGE						LONG-PERIOD RANGE				
	RETURN PERIOD (YRS)						RETURN PERIOD (YRS)				
	70	500	1000	2500	4000	NDFEF	70	500	1000	2500	4000
Vermont											
Virginia											
Langley	1.4	5.2	8.2	14.9	20.3	2.9	0.5	2.1	3.4	6.3	8.8
Washington											
Fairchild	4.8	13.2	18.8	30.2	38.5	2.3	1.6	4.2	5.9	9.3	11.8
McChord	26.1	61.5	83.2	123.9	152.0	2.0	9.1	20.2	26.8	38.9	47.1
Washington DC											
Bolling	1.8	6.5	10.2	18.4	24.9	2.8	0.6	2.3	3.6	6.5	8.9
HQ USAF	1.8	6.5	10.2	18.4	24.9	2.8	0.6	2.3	3.6	6.5	8.9
West Virginia											
Wisconsin											
Wyoming											
Francis E. Warren	2.6	7.7	11.4	19.1	24.8	2.5	0.7	2.3	3.4	5.8	7.6

[1] Expressed in percent of the acceleration of gravity. Values of spectral response acceleration from the table are divided by 100 for use

Table A2.2. Peak Ground Accelerations for Evaluation (%g)

INSTALLATION	RETURN PERIOD (YRS)				
	70	500	1000	2500	4000
Alabama					
Maxwell/ Gunter Annex	0.7	2.5	3.9	7.0	9.5
Alaska					
Eielson	12.0	19.0	38.0	49.0	57.0
Elmendorf	10.0	40.0	58.0	80.0	144.0
Arizona					
Luke	2.2	5.0	6.8	10.0	12.3
Davis Monthan	2.0	5.9	8.6	14.2	18.4
Arkansas					
Little Rock	1.3	6.9	12.3	26.4	39.1
California					
Beale	6.2	11.8	14.8	19.9	23.2
Edwards	11.6	21.6	26.9	35.9	41.7
March	32.7	52.2	61.5	76.3	85.3
McClellan	7.3	12.8	15.6	20.2	23.1
Los Angeles	23.7	44.8	56.0	75.4	87.8
Travis	23.9	47.9	61.2	84.6	99.8
Vandenberg	12.6	26.1	33.7	47.3	56.3
Colorado					
Buckley	0.8	3.1	5.0	9.2	12.7
Falcon	0.7	2.8	4.5	8.5	11.8
Peterson	1.6	3.0	3.7	5.0	5.8
USAF Academy	1.1	3.2	4.7	7.8	10.2
Connecticut					
Delaware					
Dover	0.6	2.8	4.8	9.7	13.9
Florida					
Eglin	0.3	1.4	2.3	4.7	6.7
Hurlburt Field	0.3	1.4	2.3	4.5	6.3

Table A2.2. Peak Ground Accelerations for Evaluation (%g) (Continued)

INSTALLATION	RETURN PERIOD (YRS)				
	70	500	1000	2500	4000
Florida (Cont.)					
Homestead	0.1	0.5	1.1	2.7	4.4
MacDill	0.2	0.9	1.8	4.0	6.1
Patrick	0.2	1.0	2.0	4.5	6.9
Tyndall	0.3	1.2	2.1	4.2	5.9
Georgia					
Dobbins	1.5	5.0	7.6	13.2	17.6
Robins	0.9	3.3	5.2	9.3	12.6
Moody	0.6	2.2	3.4	6.3	8.6
Hawaii					
Hickam	4.0	9.0	12.0	18.0	22.0
Idaho					
Mountain Home	3.2	8.2	11.4	17.6	22.1
Illinois					
Scott	3.0	11.3	18.1	33.9	46.7
Indiana					
Grissom	0.8	2.5	3.7	6.3	8.3
Iowa					
Kansas					
McConnell	0.5	2.1	3.5	6.9	9.7
Kentucky					
Louisiana					
Barksdale	0.5	2.3	3.8	7.5	10.6
Maine					
Maryland					
Andrews	0.7	2.7	4.3	7.9	10.8
Massachusetts					
Hanscom	1.2	5.2	8.8	17.6	25.1
Otis	0.7	3.5	6.1	12.7	18.5

Table A2.2. Peak Ground Accelerations for Evaluation (%g) (Continued)

INSTALLATION	RETURN PERIOD (YRS)				
	70	500	1000	2500	4000
Michigan					
Selfridge	0.5	1.8	2.8	5.2	7.1
Minnesota					
Mississippi					
Columbus	1.0	3.6	5.8	10.7	14.6
Missouri					
Whiteman	0.5	2.0	3.1	5.7	7.8
Montana					
Malmstrom	2.7	5.6	7.3	10.3	12.3
Nebraska					
Offutt	0.4	1.8	3.0	5.9	8.4
Nevada					
Nellis	5.1	13.7	19.4	30.7	38.9
New Hampshire					
New Jersey					
McGuire	1.0	4.8	8.3	17.1	24.8
New Mexico					
Cannon	0.4	1.7	2.7	5.0	6.9
Holloman	1.8	5.8	8.7	14.9	19.6
Kirtland	4.4	12.1	17.4	28.0	35.7
New York					
Griffiss	1.1	3.9	6.2	11.3	15.3
North Carolina					
Seymour Johnson	0.6	2.8	4.7	9.4	13.3
Pope	0.9	3.7	6.3	12.5	17.7
North Dakota					
Grand Forks	0.1	0.6	1.1	2.2	3.2
Minot	0.2	0.9	1.5	2.8	3.9

Table A2.2. Peak Ground Accelerations for Evaluation (%g) (Continued)

INSTALLATION	RETURN PERIOD (YRS)				
	70	500	1000	2500	4000
Ohio					
Rickenbacker	0.9	2.8	4.2	7.3	9.7
Wright-Patterson	0.8	3.5	5.9	11.4	16.1
Oklahoma					
Altus	0.5	2.5	4.5	9.9	14.8
Tinker	1.0	5.0	8.9	18.9	27.8
Vance	0.7	3.2	5.5	11.0	15.8
Oregon					
Pennsylvania					
Rhode Island					
South Carolina					
Charleston	2.9	19.0	36.8	87.7	136.9
McEntire	2.4	10.6	17.8	35.4	50.3
Shaw	2.2	10.4	18.2	37.8	55.0
South Dakota					
Ellsworth	0.5	2.2	3.6	6.9	9.6
Tennessee					
Arnold	1.6	5.3	8.1	14.1	18.9
Texas					
Brooks	0.2	1.3	2.6	6.7	10.8
Carswell	0.4	1.7	2.6	4.9	6.7
Dyess	0.3	1.1	1.9	3.5	4.9
Goodfellow	0.3	1.0	1.7	3.3	4.7
Kelly	0.2	1.3	2.5	6.1	9.7
Lackland	0.2	1.3	2.5	6.1	9.7
Laughlin	0.3	1.0	1.6	3.0	4.1
Randolph	0.2	1.4	2.6	6.2	9.6
Sheppard	0.5	2.3	3.9	7.9	11.4
Utah					
Hill	7.7	21.7	31.3	50.7	64.9

Table A2.2. Peak Ground Accelerations for Evaluation (%g) (Continued)

INSTALLATION	RETURN PERIOD (YRS)				
	70	500	1000	2500	4000
Vermont					
Virginia					
Langley	0.6	2.2	3.5	6.6	9.2
Washington					
Fairchild	2.3	6.1	8.6	13.6	17.1
McChord	12.9	28.1	37.0	53.2	64.1
Washington DC					
Bolling	0.7	2.7	4.3	8.1	11.1
HQ USAF	0.7	2.7	4.3	8.1	11.1
West Virginia					
Wisconsin					
Wyoming					
Francis E. Warren	0.9	3.2	5.0	9.0	12.2

SEISMIC EFFECTS AND FORCE DEMANDS

A3.1. Introduction. This attachment defines the basic seismic force demands to be used in evaluation and provides a discussion of seismic vulnerability of buildings useful in conducting evaluations and determining retrofit concepts. The discussion of seismic vulnerability of buildings is based on FEMA 172, *NEHRP Handbook for Seismic Rehabilitation of Existing Buildings* (Reference 12). FEMA 172 should be reviewed prior to developing rehabilitation concepts of buildings with deficiencies.

A3.2. Seismic Force Demands.

A3.2.1. Seismic base shear is the basic demand on the building. Element forces and deflections obtained from the approximate analysis based on this demand are the element demands, E, to be used in the prescribed load combinations. Seismic base shear equations of this ETL are different from those of FEMA 178 (Reference 4). Equations for rapid and detailed structural evaluation are in terms of the new spectral acceleration coefficients, S_{DS}, S_{DL}, S_{MS}, and S_{ML} provided in Table A2.1 (Attachment 2). In keeping with the adopted procedures of AFJMAN 32-1049 (Reference 9), the base shear equations used in detailed structural evaluation do not include the response modification coefficient, R. However, to ensure effective use of FEMA 178 evaluation statements, a revised set of base shear equations are used that retain the R coefficient.

A3.2.2. Base Shear Equations. Seismic base shear, V, in a given direction, should be determined for use in these evaluations as shown below. Computed values and supporting parameters will be recorded on a Seismic Lateral Load Calculation Data Form (Attachment 6).

$$V = C_S W \qquad \text{(A3-1)}$$

where:

C_S = the seismic design coefficient determined below

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

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live.
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 0.42 kg/m^2 (10 lb/ft^2) floor area, whichever is greater.
3. Total operating weight of all permanent equipment.
4. The effective snow load is equal to either 70 percent of the full design snow load or, where conditions warrant and approved by the Project Officer, not less than 20 percent of the full design snow load except that, where the design snow load is less than 1.26 kg/m^2 (30 lb/ft^2), no part of the load need be included in seismic loading.

A3.2.3. Seismic Coefficient, C_S, for Rapid Structural Evaluation. Rapid structural evaluations are conducted in accordance with Attachment 5. The rapid structural evaluation procedure is the same as that described in FEMA 178 with two exceptions. First, the seismic lateral force equations which follow are used. Second, in addition to the check of sufficiency as featured in FEMA 178 for the design earthquake (e.g., EQ-II), a check is made of structural sufficiency to resist without collapse the effects of EQ-III, the Collapse Limit Earthquake (CLE). The design check is made by repeating the quick check calculations redefining the design ground motions, S_{DS} and S_{DL}, as 2/3 times the maximum considered earthquake ground motion. The 2/3 factor is based on the estimated seismic margins in the design process of building code provisions. It is equal to the inverse of the limiting value of the National Design Force Exceedance Factor (NDFEF = 1.5) used to determine whether or not design validation for the large earthquake is required. Thus, a two-level procedure (Level A and Level B) is used in conducting the rapid structural evaluation. The seismic base shear, V, in a given direction, should be determined for use in these evaluations using the seismic coefficient, C_S, calculated as follows.

A3.2.3.1. Seismic Coefficient for Level A Rapid Structural Evaluation.

$$C_s = 0.85\frac{S_{adl}}{RT^n} \qquad \text{(A3-2)}$$

where:

S_{adl} = $F_v S_{DL}$
= design spectral acceleration in the long-period range for the design earthquake

S_{DL} = the spectral acceleration in the long-period range for soil profile Type B for the design earthquake ground motion representing EQ-II spectral acceleration, S_{DL}, as provided in Attachment 2

F_v = site coefficient in the long-period range given in Table A3.3 of Attachment 3 using S_{DL}=S_{DL}

R = response modification coefficient from Table A3.6 of Attachment 3

T = fundamental period of the building (paragraph A3.2.6, Attachment 3)

n = 1.0 for T<1.0 second and 2/3 for T>1.0 second

The value of C_s need not be greater than the following:

$$C_s = 0.85\frac{S_{ads}}{R} \qquad \text{(A3-3)}$$

where:

S_{ads} = $F_a S_{DS}$
= design spectral acceleration in the short-period range for the design earthquake

S_{DS} = spectral acceleration in the short-period range for soil profile Type B for the design earthquake ground motion representing EQ-II spectral acceleration, S_{DS}, as provided in Attachment 2

F_a = site coefficient in the short-period range given in Table A3.4 of Attachment 3 using S_{DS}=S_{DS}

R = response modification coefficient from Table A3.6 of Attachment 3

A3.2.3.2. Seismic Coefficient for Level B Rapid Structural Evaluation.

$$C_s = 0.57\frac{S_{avl}}{RT^n} \qquad \text{(A3-4)}$$

where:

S_{avl} = $F_v S_{ML}$
= design validation spectral acceleration in the long-period range

S_{ML} = the spectral acceleration in the long-period range for soil profile Type B for the design earthquake ground motion representing EQ-III spectral acceleration, S_{ML}, as provided in Attachment 2

F_v = site coefficient in the long-period range given in Table A3.3 of Attachment 3 using S_{DL}=2/3 S_{ML}

R = response modification coefficient from Table A3.6 of Attachment 3

T = the fundamental period of the building (paragraph A3.2.6, Attachment 3)

n = 1.0 for T<1.0 second and 2/3 for T>1.0 second

The value of C_s need not be greater than the following:

$$C_s = 0.57\frac{S_{avs}}{R} \qquad \text{(A3-5)}$$

where:

S_{avs} = $F_a S_{MS}$
= design validation spectral acceleration in the short-period range

S_{MS} = spectral acceleration in the short-period range for soil profile Type B for the design earthquake ground motion representing EQ-III spectral acceleration, S_{MS}, as provided in Attachment 2

F_a = site coefficient in the short-period range given in Table A3.4 of Attachment 3 using S_{DS}=2/3 S_{MS}

R = response modification coefficient from Table A3.6 of Attachment 3

A3.2.4. Seismic Coefficient, C_S, for Detailed Structural Evaluation. Detailed structural evaluations are conducted in accordance with Attachment 8. The seismic base shear, V, in a given direction, should be determined for use in these evaluations using the seismic coefficient, C_S, calculated as follows:

$$C_s = 0.85\frac{S_{aml}}{T^n} \qquad \text{(A3-6)}$$

where:

S_{aml} = $F_v S_{ML}$
= design validation spectral acceleration in the long-period range for the maximum earthquake ground motion considered

S_{ML} = the spectral acceleration in the long-period range for soil profile Type B for the earthquake ground motion considered representing EQ-I, EQ-II, or EQ-III spectral acceleration, S_{ML}, as provided in Attachment 2

F_v = site coefficient in the long-period range given in Table A3.3 of Attachment 3 using S_{DL}=S_{ML}

T = the fundamental period of the building (paragraph A3.2.6, Attachment 3)

n = 1.0 for T<1.0 second and 2/3 for T>1.0 second

The value of C_s need not be greater than the following:

$$C_s = 0.85\, S_{ams} \qquad \text{(A3-7)}$$

where:

S_{ams} = $F_a S_{MS}$

= design validation spectral acceleration in the short-period range for the maximum earthquake ground motion considered representing EQ-I, EQ-II, or EQ-III spectral acceleration, S_{MS}

S_{MS} = spectral acceleration in the short-period range for soil profile Type B for the maximum earthquake ground motion considered representing EQ-I, EQ-II, or EQ-III spectral acceleration, S_{MS}, provided in Attachment 2

F_a = site coefficient in the short-period range given in Table A3.4 of Attachment 3 using $S_{DS}=S_{MS}$

A3.2.5. Determination of Site Coefficients, F_a and F_v**.** The values of site coefficients F_a and F_v are based on soil profile type and ground shaking intensity.

A3.2.5.1. Soil Profile Types. The soil profile type is determined using Table A3.1. As an exception to Table A3.1, when the soil properties are not known in sufficient detail to determine the soil profile type, Type D shall be used. Soil profile Types E or F need not be assumed unless the building owner (MAJCOM or base) determines that Types E or F may be present at the site or in the event that Types E or F are established by the geotechnical data. If the $\overline{S}_u$ is used and the $\overline{N}_{ch}$ and $\overline{S}_u$ criteria differ, select the category with the softer soils (for example, use soil profile Type E instead of D).

Table A3.1. Soil Profile Type Classification

Soil Profile Type*	Soil Profile	$\overline{v}_s$	N or N_{ch}	$\overline{s}_u$
A	Hard rock	>1500 m/s (>5,000 f/s)		
B	Rock	760 to 1500 m/s (2,500 to 5,000 f/s)		
C	Very dense soil and soft rock	360 to 760 m/s (1,200 to 2,500 f/s)	>50	≥100 kPa (≥2,000 lb/ft^2)
D	Stiff soil	180 to 360 m/s (600 to 1,200 f/s)	15 to 50	50 to 100 kPa (1,000 to 2,000 lb/ft^2)
E	Soil	<180 m/s (<600 f/s) or any profile with more than 3 m (10 ft) of soft clay defined with PI>20, w≥40 percent, and s_u <25 kPa (500 lb/ft^2)	<15	<50 kPa (<1,000 lb/ft^2)
F	A soil profile requiring site-specific evaluations: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H>3 m [10 ft] of peat and/or highly organic clay where H=thickness of soil) 3. Very high plasticity clays (H>8 m [25 ft] with PI>75) 4. Very thick soft/medium stiff clays (H>36 m [120 ft])			

*These soil types are defined in the 1994 *NEHRP Recommended Provisions* (Reference 8).

A3.2.5.2. Steps for Classifying a Site (also see Table A3.2 below):

Step 1: Check for the four categories of soil profile Type F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as soil profile Type F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay 3 meters (>10 feet) where a soft clay layer is defined by: $\overline{S}_u$ <25 kPa (500 lb/ft^2), w$\geq$40 percent, and PI>20. If these criteria are satisfied, classify the site as soil profile Type E.

Step 3: Categorize the site using one of the following three methods with $\overline{v}_s$, $\overline{N}$, and $\overline{S}_u$ computed in all cases as specified by the definitions in paragraph A3.2.5.3:

a. $\overline{v}_s$ for the top 30 meters (100 feet) (v_s method)

b. $\overline{N}$ for the top 30 meters (100 feet) ($\overline{N}$ method)

c. $\overline{N}_{ch}$ for cohesionless soil layers (PI<20) in the top 30 meters (100 feet) and average $\overline{S}_u$ or cohesionless soil layers (PI>20) in the top 30 meters (100 feet) ($\overline{S}_u$ method)

Table A3.2. Soil Profile Type Classification

Soil Profile Types*	Soil Profile	$\overline{v}_s$	$\overline{N}$ or $\overline{N}_{ch}$	$\overline{S}_u$
C	Very dense soil and soft rock	360 to 760 m/s (1,200 to 2,500 f/s)	>50	≥100 kPa (≥2,000 lb/ft^2)
D	Stiff soil	180 to 360 m/s (600 to 1,200 f/s)	15 to 50	50 to 100 kPa (1,000 to 2,000 lb/ft^2)
E	Soil	<180 m/s (<600 f/s)	<15	<50 kPa (<1,000 lb/ft^2)

NOTE: If the s_u method is used and the $\overline{N}_{ch}$ and $\overline{S}_u$ criteria differ, select the category with the softer soils (for example, use soil profile Type E instead of D).

A3.2.5.2.1. The shear wave velocity for rock, soil profile Type B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall be measured on site for shear wave velocity or classified as soil profile Type C.

A3.2.5.2.2. The hard rock, soil profile Type A category, shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 meters (100 feet), superficial shear wave velocity measurements may be extrapolated to assess $\overline{v}_s$.

A3.2.5.3. Definitions. The definitions presented below apply to the upper 30 meters (100 feet) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 30 meters. The symbol I then refers to any one of the layers between 1 and n.

v_{si} is the shear wave velocity in m/s (f/s)

d_i is the thickness of any layer between 0 and 30 meters (100 feet)

$\overline{v}_s$ is:

$$\overline{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \text{(A3-8)}$$

where $\sum_{i=1}^{n} d_i$ is equal to 30 meters (100 feet).

N_i is the standard penetration resistance (ASTM D1586-84) not to exceed 305 blows per meter (100 blows per foot) as directly measured in the field without corrections.

$\overline{N}_i$ is:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{(A3-9)}$$

$\overline{N}_{ch}$ is:

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N}} \qquad \text{(A3-10)}$$

where:

$\sum_{i=1}^{m} d_i$ = d_s. Use only d_i and N_i for cohesionless soils.

d_s is the total thickness of cohesionless soil layers in the top 30 meters (100 feet).

s_{ui} is the undrained shear strength in kilopascals (pounds per square foot), not to exceed 250 kilopascals (5,000 pounds per square foot) (ASTM D2166-91 or D2850-87).

$\overline{s}_u$ is:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad \text{(A3-11)}$$

where:

$$\sum_{i=1}^{k} d_i = d_c.$$

d_c is the total thickness (100 - d_S) of cohesive soil layers in the top 30 meters (100 feet).

PI is the plasticity index (ASTM D4318-93).

w is the moisture content in percent, ASTM D2216-92.

A3.2.5.4. Site Coefficients F_v and F_a**.** The site coefficients F_v and F_a are as indicated in Tables A3.3 and A3.4, respectively, and are used to determine the seismic coefficient, C_s**.**

Table A3.3. Values of F_v For Class B Sites

Site Class	Design Spectral Acceleration at 1 Second for Class B Sites				
	$S_{DL} \leq 0.1$	$S_{DL}=0.2$	$S_{DL}=0.3$	$S_{DL}=0.4$	$S_{DL} \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a
F	a	a	a	a	a

NOTE: Use straight line interpolation for intermediate values of S_{DL}. Site-specific geotechnical investigation and dynamic site response analysis shall be performed.

Table A3.4. Values of F_a For Class B Sites

Site Class	Design Spectral Acceleration at Short Periods for Class B Sites				
	$S_{DS} \leq 0.25$	$S_{DS}=0.50$	$S_{DS}=0.75$	$S_{DS}=1.00$	$S_{DS} \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a
F	a	a	a	a	a

NOTE: Use straight line interpolation for intermediate values of S_{DS}. Site-specific geotechnical investigation and dynamic site response analysis shall be performed.

A3.2.6. Period. For use in Equations A3-2 and A3-6, the value of T should be calculated using one of the following methods.

A3.2.6.1. 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.

A3.2.6.1.1. 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,

$$T_a = C_T h_n^{3/4} \qquad \text{(A3-12)}$$

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

A3.2.6.1.2. As an alternate for concrete and steel frame buildings of 12 stories or fewer with a minimum story height of 3 meters (10 feet), the equation T_a = 0.010N, where N = the number of stories, may be used in lieu of Equation A3-12.

A3.2.6.1.3. For all other buildings:

$$T_a = \frac{0.05 h_n}{\sqrt{L}} \qquad \text{(A3-13)}$$

where:

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

A3.2.6.2. 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:

$$T = 2p\sqrt{\frac{\Sigma\left(w_i d_i^2\right)}{g\Sigma\left(f_i d_i\right)}} \qquad \text{(A3-14)}$$

A3.2.6.2.1. The values of f_i represent any lateral force, associated with weights w_i, distributed approximately in accordance with Attachment 8, paragraph A8.10.3, 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 A3.5.

Table A3.5. Coefficient for Upper Limit on Calculated Period

S_{adl}*	C_a
0.60	1.2
0.45	1.3
0.30	1.4
0.20	1.5

0.15	1.7
0.075	1.8

Table A3.6 Response Coefficients*

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, non-moment 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

Table A3.6 Response Coefficients* (Continued)

R	C_d	System
		Dual System with a Special Moment Frame Capable of Resisting at Least 25% 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, non-moment 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% 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

*The response modification factors, (R), and deflection amplification factors, (C_d), are from Table 3-2 of the 1991 *NEHRP Recommended Provisions* (Reference 8). See these provisions for details.

A3.3. Seismic Vulnerability of Buildings. This section describes the general characteristics of all structural materials and systems (i.e., strength, stiffness, ductility, and damping) and the design and construction features that may adversely affect the seismic performance of a structure. Vulnerability assessment considers these characteristics. An informed decision regarding the most cost-effective techniques for rehabilitating an existing structure to resist seismic forces requires an understanding of the structural system or combination of systems that resist the lateral loads, the advantages or disadvantages associated with the physical attributes of the systems, and the constraints on system performance due to adverse design or construction features. Hence, the emphasis here is on the complete structural system.

A3.3.1. General Attributes of Structures. Strength, stiffness, ductility, and damping govern the dynamic response of a structure to ground motion. An ideal structure would rate highly with respect to all of these attributes; however, this is seldom the case even in new construction and may be impossible to achieve when strengthening an existing structure. Fortunately, these attributes are interrelated, and it is usually possible to compensate for a deficiency in one by enhancing one or more of the others (e.g., additional strength and stiffness may compensate for low ductility and damping).

A3.3.1.1. Strength. The most obvious, although not necessarily the most important, consideration in seismic rehabilitation is strength. A seismically weak structure can be rehabilitated by strengthening existing members or by adding new members that increase the overall strength of the structure. Many of the rehabilitation techniques presented in FEMA 172 are aimed at increasing strength, and informed identification of the building elements that should be strengthened can lead to significant cost savings in an upgrading scheme.

A3.3.1.2. Stiffness. As indicated by the base shear formula in the 1991 *NEHRP Recommended Provisions* (Reference 8), structural stiffening that reduces the fundamental period of the building may result in higher seismic forces to be resisted by the building. Nonetheless, additional stiffening generally will reduce the potential for seismic damage. Drift limitations specified by most building codes are intended to provide for minimum structural stiffness.

A3.3.1.2.1. Transfer of loads among the elements of a structure depends on the relative stiffness of those elements. To select the most appropriate technique for seismically rehabilitating a structure, it is important to evaluate the stiffness of both the existing elements and those to be added to ensure that the seismic load path is not altered in a way that creates new problems. To contribute effectively, an added element must be stiff enough relative to the existing lateral-force-resisting elements to attract sufficient load away from the existing system. The location of an added member and, therefore, the added stiffness it contributes, also is important. The engineer should attempt to locate new elements in such a way as to minimize eccentricities in the building and limit torsional responses.

A3.3.1.3. Ductility. The ductility of a structure or element (i.e., the ability of the structure or element to dissipate energy inelastically when displaced beyond its elastic limit without a significant loss in load carrying capacity) is an extremely important consideration in seismic rehabilitation. The structural properties of some materials have a post-elastic behavior that fits the classic definition of ductility (i.e., they have a near-plastic yield zone and this behavior is reasonably maintained under cyclic loading). Other materials, such as reinforced concrete and masonry, nailed wood systems, braced frames, and floor diaphragms, have stiffness degradation and may even exhibit a pinched load-displacement relationship when subjected to cyclic loading. The hysteretic damping of these materials may not increase as is common for the elastic-plastic behavior but the stiffness degradation has a beneficial influence similar to an increase in damping in that the base shear of the system is reduced. However, the interstory and total relative displacement of the stiffness degrading structure or element is significantly increased. Control of relative displacement of this class of structure or element is of prime importance.

A3.3.1.4. Damping. During an earthquake, a structure will amplify the base ground motion. The ground motion at the base includes the amplification caused by soil profile type through the inclusion of a soil profile coefficient in the base shear formula. The degree of structural amplification of the ground motion at the base of the building is limited by structural damping or the ability of the structural system to dissipate the energy of the earthquake ground-shaking. The differences in the response modification coefficient, (R), and the deflection amplification factor, (C_d), of Table 3-2 of the 1991 *NEHRP Recommended Provisions* (Reference 8), are partially due to an estimation of probable structural damping of greater than five percent of critical.

A3.3.2. Adverse Design and Construction Features. A number of design and construction features have an adverse impact on structural response by precluding the effective development of the capacity of the various structural components.

A3.3.2.1. Lack of Direct Load Path. An adequate load path is the most essential requirement for seismic resistance in a building. There must be a lateral-force-resisting system that forms a direct load path between the foundation, the vertical elements, and all diaphragm levels, and that ties all portions of the building together. The load path must be complete and sufficiently strong. The general path is as follows.

- Earthquake inertia forces, which originate in all elements of a building, are delivered through structural connections to horizontal diaphragms.
- The diaphragms distribute these forces to vertical components of the lateral-force-resisting system such as shear walls and frames.
- The vertical elements transfer the forces into the foundation.
- The foundation transfers the forces into the ground.

The load path therefore consists of elements within and between the following subsystems: vertical-resisting elements, diaphragms, and foundations.

A3.3.2.2. Irregularities. Most building codes prescribe seismic design forces that are only a fraction of the forces that would be imposed on a linearly elastic structure by a severe earthquake. These codes therefore imply that the inelastic response of the designed structures is required to fulfill the primary performance objective (i.e., preserve life safety by precluding structural collapse). The equivalent static lateral loads and design coefficients prescribed by the codes are necessarily imperfect approximations of the nonlinear dynamic response of code-designed regular structures. Vertical and plan irregularities can result in loads and deformations significantly different from those assumed by the equivalent static procedures. It is most important for the engineer to understand that severe irregularities can create uncertainties in the ability of the structure to meet the stated performance objectives. Irregular conditions exist, to some degree, in most buildings. Minor irregularities have little or no detrimental effect on structural response. Guidelines for the evaluation of the significance of the vertical and horizontal or plan irregularities are provided in the NEHRP Evaluation Handbook (Reference 4). If a significant irregular condition cannot be avoided or eliminated by design changes, the designer should both comply with any special provisions prescribed by the code and consider the ability of the structure to avoid collapse when subjected to relative displacements that may be several times greater than the anticipated nonlinear displacements.

A3.3.2.2.1. Vertical Irregularities. The vertical irregularities that may adversely affect a building's seismic resistance are discussed briefly. Stiffness irregularity results when one or more stories are significantly softer (i.e., will be subject to larger deformations) than the stories directly above. Weight or mass irregularity occurs when the effective mass (i.e., weight divided by the acceleration due to gravity) of any story is substantially greater than the effective mass of an adjacent story. Vertical geometric irregularity results from building setbacks or elevational discontinuities (i.e., when the upper portions of a building are reduced in plane area with respect to the lower portions). Vertical discontinuity in capacity occurs when the story strength in a story is significantly less than that in the story above. The story strength is defined as the total strength of all the seismic-resisting elements sharing the story shear for the direction under consideration. Vertical discontinuity in load path is a condition where the elements resisting lateral forces (i.e., moment frames, shear walls, or braced frames) are not continuous from one floor to the next. Figure A3.1 shows two common examples. The upper sketch shows an "out-of-plane" vertical discontinuity that causes the vertical load path to be discontinuous. In the upper sketch, the shear walls of the second and third stories are exterior shear walls, while the shear walls in the first floor are interior walls. The seismic forces from the top two stories must be transferred through the second floor diaphragm and then into the first floor shear wall. The discontinuity results in very high forces on the diaphragm. The lower sketch in Figure A3.1 is an example of an in-plane discontinuity with a potential for overturning forces in excess of the capacity of the column. The usual deficiency in the diaphragm is inadequate shear capacity. Unlike typical floor diaphragms that need only transfer tributary seismic floor shears, the diaphragm at the base of a discontinuous shear wall must transfer the cumulative seismic shears in the shear wall from all of the levels above the discontinuity. A typical cause of distress in concrete columns at the ends of discontinuous shear walls is inadequate capacity to resist the overturning loads from the discontinuous wall above. For many years, seismic provisions in building codes have prescribed factored design loads for shear walls that were in excess of those required for columns. Thus, in a severe earthquake, the discontinuous shear wall was capable of generating overturning forces in excess of the capacity of the supporting columns. During the 1979 Imperial County earthquake in California, the six-story County Services building was irreparably damaged when a number of the first story columns under discontinuous shear walls collapsed due to excessive overturning forces. As a result of that earthquake, current code provisions discourage vertical discontinuities and require special strengthening of columns if the discontinuities cannot be avoided.

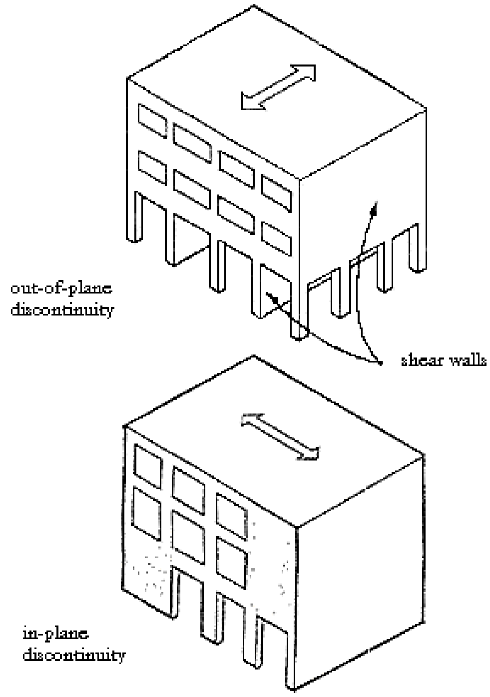


Figure A3.1. **Vertical Irregularities--Examples of In-Plane and Out-of-Plane Discontinuities**

A3.3.2.2.2. Rehabilitation Techniques for Vertical Irregularities. The obvious remedial technique for any irregularity is to modify the existing structural elements or add new structural elements to eliminate or significantly reduce the irregularity. The engineer must take special care to avoid creating greater or new problems in the existing elements. For example, if vertical bracing is used to increase the strength of a weak story, it is important to determine the effect that these modifications will have on the story stiffness (i.e., whether it will create a soft story condition in the stories below), whether it will create significant torsional eccentricity and/or whether the load path in the diaphragms above and below will be adequate for the revised distribution and transfer of the shear forces. If a new shear wall is added in a shear wall building to increase story strength or stiffness, the same concerns must be investigated. Extending the new shear wall to the foundation level is one way to avoid the vertical discontinuity. Vertical supports below the wall also should be investigated to determine their capacity to resist realistic overturning forces.

A3.3.2.2.2.1. It may not be feasible to eliminate or reduce some weight or mass irregularities (e.g., a heavy boiler extending through several stories of an industrial building) or elevational irregularities (e.g., building setbacks). If the irregularity cannot be eliminated or significantly reduced, a dynamic analysis that will better represent the structural response may be required to identify the appropriate location for needed strengthening and its extent.

A3.3.2.2.2.2. A common technique for improving the seismic performance of structures with vertical discontinuities in load path is to strengthen the columns below the discontinuity so that they can resist the vertical forces that can be imposed by overturning moments if the above walls. The diaphragm spanning between the discontinuous vertical-resisting elements also may require strengthening. Alternatively, the discontinuity can be eliminated if new vertical-resisting elements are built directly below the existing vertical-resisting elements; however, the effect the new members will have on the functional space of the building must be evaluated.

A3.3.2.2.3. Horizontal or Plan Irregularities. Plan structural irregularities in buildings that may adversely affect a building's seismic resistance are discussed briefly below. Torsional irregularity occurs in buildings with rigid diaphragms when the center of mass in any story is eccentric with respect to the center of rigidity of the vertical lateral-load-resisting elements. Nominal eccentricity, or torsion, is common in most buildings and many building codes require that an accidental eccentricity (usually prescribed as five percent of the maximum plan dimension) be added to the actual computed eccentricity to determine the torsional forces. An exception occurs when a floor or roof diaphragm is relatively flexible with respect to the vertical lateral-load-resisting elements (e.g., a nailed wood diaphragm in a building with concrete or masonry shear walls). In this case, the vertical elements are assumed to resist only tributary seismic loads. Note that by making this assumption the effects of torsion may be neglected. In some cases (e.g., steel floor or roof decking in a building with steel moment frames), the relative rigidity of the diaphragm may be difficult to assess and the designer may elect to distribute the seismic loads on the basis of a rigid diaphragm and by tributary area and then to use the more conservative results from the two methods. Re-entrant corners in the plan configuration of an existing structure (and its lateral force resisting system) create excessive shear stresses at the corner. Diaphragm discontinuity occurs when a diaphragm has abrupt discontinuities or variations in stiffness. A common diaphragm discontinuity is split level floors. Unless proper members exist either to transfer the diaphragm forces between the split levels or to independently transfer the forces via vertical members to the foundation, damage is likely to occur at the interface. This condition also exists when diaphragms have large cutout or open areas or substantial changes in effective diaphragm stiffness from one story to the next. Nonparallel systems is the condition that occurs when the vertical lateral force resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting system.

A3.3.2.2.4. Rehabilitation Techniques for Horizontal Irregularities. The seismic rehabilitation of a structure with a large eccentricity, due either to the distribution of the vertical resisting elements or the distribution of the mass in the building, is best accomplished by reducing the eccentricity. Locating stiff resisting elements that reduce the eccentricity (Figure A3.2) reduces the forces and stresses due to torsion and increases the lateral force resisting capacity of the entire structure. The seismic deformations of the entire structure also are significantly reduced by strategically locating the new walls to minimize torsion. The most direct rehabilitation technique for excessive shear stress at a re-entrant corner is to provide drag struts to distribute the local concentrated forces into the diaphragm (Figure A3.3). Other alternatives include strengthening the diaphragm with overlays and reducing the loads on the diaphragm by providing additional vertical resisting elements. Diaphragm discontinuities due to abrupt changes in stiffness can be improved by developing a gradual transition through selective stiffening of the diaphragm segments adjacent to the stiff elements. Stress concentrations in the diaphragm at the corners of large openings can be reduced by providing collector members or drag struts to distribute the forces into the diaphragm. Improving deficient conditions caused by diaphragm discontinuities (such as may be present in split level framing) can be accomplished by providing adequate load path for the lateral forces. Structures with nonparallel systems can be strengthened by ensuring that there is an adequate load path for various force components resulting from the diaphragm to the vertical lateral load resisting systems. A structure with a nonparallel system is shown in Figure A3.4. Providing a drag strut at the corner as indicated will distribute into the diaphragm the out-of-plane force component at the intersection of the two shear walls.

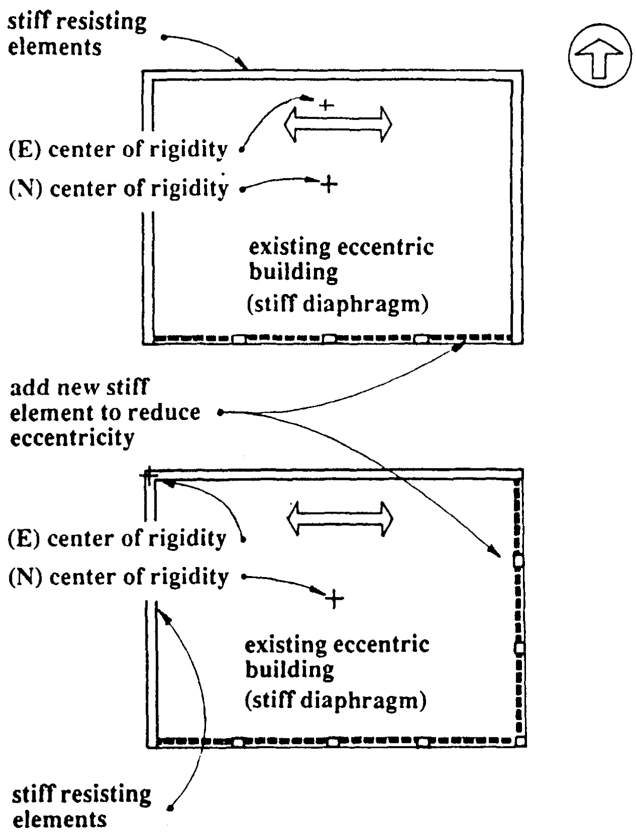


Figure A3.2. Horizontal or Plan Irregularities -- Rehabilitating a Structure to Reduce Torsion Loads

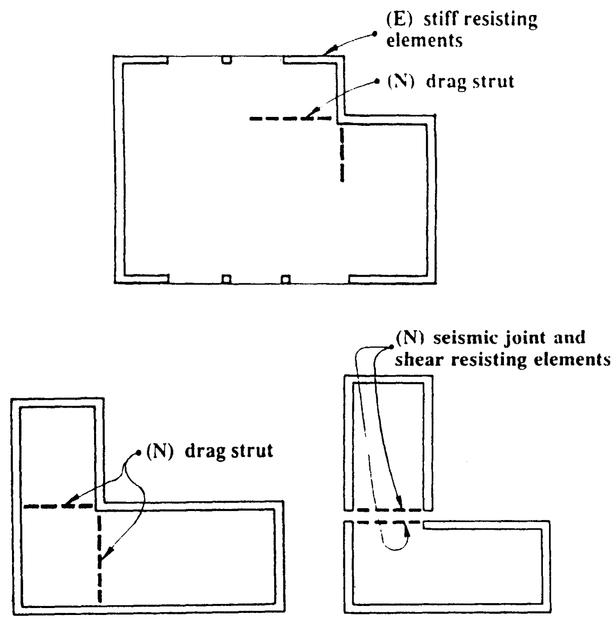


Figure A3.3. Horizontal or Plan Irregularities -- Rehabilitating Buildings With Re-entrant Corners

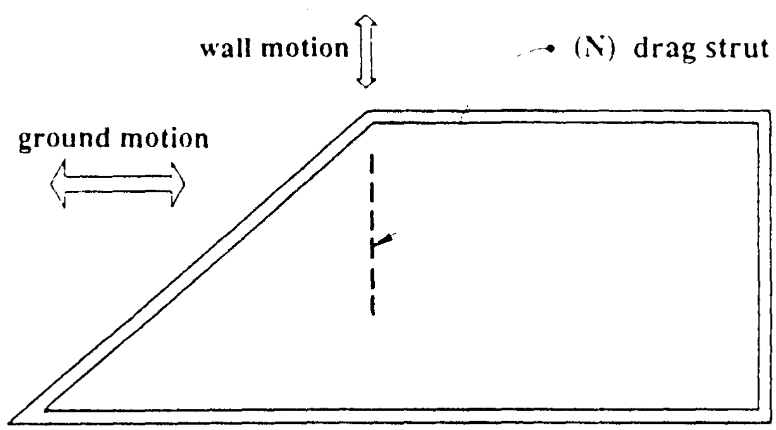


Figure A3.4. Horizontal or Plan Irregularities -- Example of Strengthening a Split Level Diaphragm

A3.3.2.2.5. Reduction of Irregularities and Re-Analysis. The irregularities discussed above will affect the dynamic response of a structure to seismic ground motion and may invalidate the approximation made in the code-prescribed equivalent static lateral force analysis. The evaluation statements (FEMA 178, Appendixes A and B) present thresholds at which these effects may be considered significant but they are necessarily subjective and should be used with judgment, particularly when a structure has more than one of the above irregularities. Although a linear elastic dynamic analysis will help to identify the location and extent of the irregular responses, any analysis is subject to the validity of the model and, for an existing structure, there may be many uncertainties in the modeling assumptions. Also, as indicated above, the uncertainties associated with the extrapolation of results of linear elastic analyses to obtain estimates of nonlinear response increase greatly when the structure is highly irregular or asymmetrical. For these reasons, structural modifications associated with seismic upgrading of an irregular building should aim primarily to eliminate or significantly reduce the irregularity. The illustration in the lower portion of Figure A3.3 is an example of an irregular building divided into two separate, regular structures by providing a seismic separation joint. This concept requires careful structural and architectural detailing at the separation joint and may not be cost-effective as a retrofit measure except in cases where extensive alterations are planned for other reasons (e.g., an industrial structure being converted to light commercial or residential use).

A3.3.2.3. Lack of Redundancy.

A3.3.2.3.1. The Problem. Structures that feature multiple load paths are said to be redundant. Loads producing temporary seismic overstress of individual members or connections in a redundant structure may be redistributed to alternate load paths with the capacity to resist these seismic loads. The seismic capacity of structures that lack redundancy is dependent on adequate nonlinear behavior of the lateral-load-resisting elements. Engineering judgment should be used to ascertain the need of redundancy.

A3.3.2.3.2. Rehabilitation Techniques for Lack of Redundancy. Rehabilitation techniques that enhance redundancy generally involve the addition of new lateral load resisting elements or new systems to supplement existing weak or brittle systems. For example, the addition of new steel braced frames or reinforced concrete shear walls in an existing concrete frame building will provide redundancy to the existing system. The relative rigidity of the new systems probably will dictate that little or none of the design lateral loads be resisted by the existing concrete frame, but if the new braced frames or shear are properly designed for ductile behavior as they yield in a severe earthquake, the lateral loads will be redistributed to take advantage of the capacity of the existing concrete frames. The example illustrates that ductility and an adequate load path are essential to the redistribution of loads in redundant systems.

A3.3.2.4. Lack of Toughness.

A3.3.2.4.1. The Problem. Toughness is defined here as the ability of a structure to maintain its integrity and preclude collapse during a severe earthquake that may cause significant structural damage.

A3.3.2.4.2. Rehabilitation Techniques for Lack of Toughness. Existing connection details and those for new structural modifications should be evaluated for toughness. The engineer must further evaluate these connections in terms of their performance under extreme structural loads and deformations. Codes may prescribe that some precautions be taken (e.g., oversizing connection requirements to avoid premature failure of bracing members that are not part of the lateral load resisting system); however, other considerations (e.g., avoiding weld configurations that could lead to prying action or other stress concentrations) require engineering judgment. For some structural systems (e.g., steel moment frames), providing additional strength in the connections will increase the toughness of the system; however, in other systems (e.g., concrete moment frames), lack of toughness may require displacement control through the addition of stiffer elements or supplemental damping to protect the existing system.

A3.3.2.5. Adjacent Buildings.

A3.3.2.5.1. The Problem. When the gap between buildings is insufficient to accommodate the combined seismic deformations of the buildings, both may be vulnerable to structural damage from the "pounding" action that results when the two collide. This condition is particularly severe when the floor levels of the two buildings do not match and the stiff floor framing of one building impacts the more fragile walls of the adjacent building.

A3.3.2.5.2. Rehabilitation Techniques for Potential Impact from Adjacent Buildings. Since the gap between two buildings usually cannot be increased, increasing the stiffness of one or both buildings may reduce the seismic deformations to the point where impact is precluded with the existing gap. This technique, however, may not be feasible for stiff shear wall buildings of concrete or masonry and, for those cases, consideration should be given to providing alternative load paths for the vertical load-resisting members (i.e., bearing walls or columns) that may be damaged or destroyed by the impact. These alternative load paths would include supplementary columns or vertical shoring to support the floor or roof systems. These supplementary supports would be installed at sufficient distance from the vulnerable exterior walls or columns to be protected when the existing elements are damaged.

A3.4. Deteriorated Condition of Structural Materials.

A3.4.1. The Problem. Structural materials that are damaged or seriously deteriorated may have an adverse effect on the seismic performance of an existing building during a severe earthquake. The significance of the damage or deterioration must be evaluated with respect to both the existing condition and the proposed seismic strengthening of the building.

A3.4.1.1. Timber. Common problems with timber members that require rehabilitation include termite attack, fungus ("dry rot" or "damp rot"), warping, splitting, checking due to shrinking, strength degradation of fire-retardant plywood in areas with high temperatures, or other causes.

A3.4.1.2. Unreinforced Masonry. The weakest element in older masonry usually is the mortar joint, particularly if significant amounts of lime were used in the mortar and the lime was subsequently leached out by exposure to the weather. Thus, cracks in masonry walls caused by differential settlement of the foundations or other causes generally will occur in the joints; however, well-bonded masonry occasionally will crack through the masonry unit.

A3.4.1.3. Unreinforced Concrete. Unreinforced concrete may be subject to cracking, spalling, and disintegration. Cracking may be due to excessive drying shrinkage during the curing of the concrete or differential settlement of the foundations. Spalling can be caused by exposure to extreme temperatures or the reactive aggregates used in some western states. Disintegration or raveling of the concrete usually is caused by dirty or contaminated aggregates, old or defective cement, or contaminated water (e.g., water with a high salt or mineral content).

A3.4.1.4. Reinforced Concrete or Masonry. Reinforced concrete and masonry are subject to the same types of deterioration and damage as unreinforced concrete and masonry. In addition, poor or cracked concrete or masonry may allow moisture and oxygen to penetrate to the steel reinforcement and initiate corrosion. The expansive nature of the corrosion byproducts can fracture the concrete or masonry and extend and accelerate the corrosion process.

A3.4.1.5. Structural Steel. Poorly designed structural steel members may trap moisture from rainfall or condensation under conditions that promote corrosion and subsequent loss of section for the steel member. Even well-designed steel members exposed to a moist environment require periodic maintenance (i.e., painting or other corrosion protection) to maintain their effective load bearing capacity. Light structural steel members (e.g., small columns or bracing members) in some installations may be subject to damage from heavy equipment or vehicles. While such damage may have no apparent detrimental effect on the vertical load resisting capacity of the steel member, its reserve capacity for resisting seismic forces may be seriously impaired.

A3.4.2. Rehabilitation Techniques for Deteriorated Condition of Structural Materials. Structural materials that exhibit evidence of damage or deterioration require careful evaluation. Even if affected structural elements are to be rehabilitated or replaced, it is important that the factors contributing to the damage or deterioration be eliminated or minimized. For example, vulnerable steel framing can be protected from heavy equipment or vehicles by concrete curbs or concrete encasement, poorly drained steel members and connections can be modified or replaced so as to provide positive drainage, and steel framing in most environments can be painted or covered with other corrosion resistant coatings. If the deterioration is not severe and the apparent causes have been mitigated, the engineer may decide to assign a reduced capacity to the structural member and to perform a revised evaluation of the need for rehabilitation and/or strengthening.

WIND EFFECTS AND FORCE DEMANDS

A4.1. Introduction and Purpose.

A4.1.1. The evaluation of buildings for life-safety must address all of the load demands which may reasonably be placed on the buildings during their life expectancy. The loads critical to the performance of buildings include the lateral wind and seismic forces as well as the more commonly recognized gravity loads (dead load and live load). Depending on the location and other factors, either wind or seismic loads may govern the forces in any structure or portion of a structure. Wind loads can be especially severe in hurricane-prone regions such as the U.S. Gulf and Atlantic coasts, Hawaii, Puerto Rico, Guam, and American Samoa. Hurricanes Hugo (1989), Andrew (1992), and Iniki (1992) demonstrated the severity of wind by causing damage levels far exceeding losses due to recent earthquake experience. Therefore, as a companion to seismic evaluation, wind evaluation is equally important to total building performance and life-safety.

A4.1.2. The procedures described for wind evaluation are intended to provide the engineer guidance on how to identify buildings or building components which pose a risk to human life or property. The objectives of the wind evaluation are not met if a building collapses, portions of the building collapse, components of the building fail, or exit routes are blocked and evacuation of the building is prevented.

A4.2. Scope and Limitations.

A4.2.1. This wind evaluation procedure deals principally with life-safety objectives; it does not address other objectives of code compliance or damage control. The wind forces applied are code-required forces as used in building design equations with allowable stresses or load factors which lead to structural capacities (resistances) substantially higher than required to resist the code-required wind speeds and associated forces. This is different from the approach used for seismic design, where actual earthquake forces and deflections may be larger than code forces and deflections, but a building will survive by dissipating energy in the yielding of its components if the code provisions concerning force level and detailing have been applied properly.

A4.2.2. For wind evaluations using the procedures of this ETL, the demand is based upon the code-required wind speed, pressures, and forces. See Table 3, Performance Requirements for Wind, in this ETL. The criteria for acceptance under these evaluation procedures will differ from code criteria as follows.

A4.2.2.1. High Risk and Other Buildings (Performance Objective Category III and IV). For these buildings, the code level demand will be evaluated against an ultimate-strength capacity basis obtained by using the procedures of the material chapters of the NEHRP *Recommended Provisions* (Reference 8), and FEMA 178, Sec. 2.4.9, (Reference 4); i.e., converting to nominal strengths by multiplying working stresses by factors given for the various materials (for example, ASD allowable stresses times 1.7 for steel) without using the capacity reduction factor, ϕ.

A4.2.2.2. Immediate Occupancy Buildings (Performance Objective Category I). Buildings in this category will be evaluated using code level demand and code level capacity; i.e., allowable stresses increased by one-third for wind or factored wind loads for strength design.

A4.2.3. As a general rule, the evaluation statements of FEMA 178, Appendixes A and B (Reference 4), which are to be completed for the seismic evaluation, will also address important concerns for the wind evaluation. Supplemental wind evaluation statements are included in this ETL to address concerns unique to the wind environment and respond to quick check evaluations. Quick check procedures may be similar to the seismic quick check procedures, but require separate evaluation for the wind load. The wind evaluation included in this ETL will be limited to the Main Wind-Force Resisting System (MWFRS) and those components essential to the stability of the building. This wind evaluation will not include other components and cladding which, if failure occurs, may expose the building to serious wind and water damage to such nonstructural elements as finishes and contents.

A4.2.4. The "Basic Wind Speed" is used to determine the design wind pressures and forces. The wind speed map of the contiguous United States (Figure 1, ASCE 7-95) (Reference 3) gives "fastest-mile wind speeds" at 10 meters (33 feet) above the ground for exposure Category C based on an annual probability of 0.02 that the wind speed is exceeded (50-year mean recurrence interval). Tornadoes have not been considered in developing the basic wind speed distributions. Sufficient information is available to implement tornado-resistant design for above-ground shelters and for buildings that house essential facilities. For those buildings that must be designed to resist tornadic winds, refer to ASCE 7-95 Commentary C, paragraph 6.5.2.3. Similarly, special consideration should be given to those regions for which records or experience indicates that the wind speeds are higher than those reflected in Figure 1 (see ASCE 7-95, paragraph 6.5.2.1).

A4.3. Wind Forces and Effects on Buildings.

Note: Paragraphs A4.3 through A4.7.5.2 have been adapted from Appendix F of ATC-26-2, *Procedures for Postdisaster Safety Evaluation of Postal Service Facilities* (Interim) (Reference 18). Appendix F was developed by Lindbergh & Associates in consultation with Dr. Dale C. Perry and Dr. W. Lynn Beason.

A4.3.1. To evaluate buildings subjected to wind loads for life-safety, it is important to understand the nature of wind and the forces wind exerts on buildings in its path. Armed with this knowledge, the building evaluator can deduce, to some extent, how different building elements would be affected by the wind and what kind of damage to expect. In this way, the effectiveness of the evaluator and the quality of the evaluation are greatly improved. This introduction to wind life-safety risks includes a simplified technical discussion of the wind forces as well as examples of typical wind damage.

A4.3.2. The forces imposed by an earthquake are different from the forces imposed by wind. As a result, the type of building damage caused by an earthquake will be different than the type of damage to the same building resulting from wind. Therefore, it is appropriate to establish evaluation criteria and procedures that are event-specific. These evaluation procedures can then be used concurrently, as the situation dictates, to better evaluate the safety of a building. There are two major differences that should be considered when evaluating buildings for wind rather than earthquake loads.

A4.3.2.1. First, there is the fundamental difference in how wind and earthquake lateral loads are transferred to a building. The earthquake load is transmitted to the building from its foundation. In general, the intensity of the forces experienced by particular building elements is proportional to their respective masses. Consequently, the entire building and its contents will experience the force. On the other hand, wind loads are transmitted to the building through its exterior envelope. The cladding and its supporting members experience the initial effects of the wind. Except for supporting structural members, the interior of the building, including its contents, will not directly experience the wind loads as long as the exterior envelope remains intact. During Hurricane Hugo, the interior unreinforced masonry partitions on many buildings went undamaged despite significant damage to their exterior structures. In contrast, many interior nonload bearing masonry partitions were heavily damaged through lateral movement by the Prince William Sound, Alaska, earthquake of 1964.

A4.3.2.2. A second major difference is in the degree of anticipated damage. When the building has been designed for wind loads according to building code provisions, the building is expected to perform entirely within the elastic limit of its materials. The building is expected to resist such loads without damage. However, in the case of an earthquake, the building is expected to experience actual forces and ground displacements much greater than those associated with the design earthquake loads. This maintains elastic design forces at reduced levels, taking advantage of the ultimate capacity of the structure to resist more severe load conditions by deforming beyond the elastic range. This difference in design concept is commonly reflected in the nature of wind and earthquake building damage expected.

A4.3.3. Buildings are subjected to the forces of wind on a continuing basis. Generally, these wind forces are at levels well within the capability of the structure to resist them, whether that capability is based on an engineered design using building code-specified wind loads; or, as in the case of residential construction, on standard construction practices that have developed over time. Periodically, structures are subjected to wind forces that cause damage. In some instances, the damage is due to wind loads exceeding design criteria. In others the damage results from a weakness (design or construction deficiency) in the building. The performance of most engineered and properly constructed buildings subjected to near design-level winds has validated the technology currently used.

A4.3.4. Damaging wind forces are usually associated with extreme weather phenomena, such as tornadoes, hurricanes, or thunderstorms. Maps indicating wind velocities for 100- or 50-year recurrences have been used in building codes to establish wind loads for building design. The maps and other factors in design standards take into account the varying wind loads experienced in different environments; i.e., near the coast, inland open terrain, and urban environment.

Although design wind loads are expressed in terms of a sustained or average wind velocity, building codes and standards generally use gust factors to account for the effects of wind gusts that exceed the sustained wind velocity. The actual wind loads on a building rarely exceed the design wind load. Even in cases where design level winds are exceeded, the well-designed and constructed building probably will sustain relatively little damage.

A4.3.5. Many buildings would suffer severe damage if struck directly by a tornado. This damage results not only from the extreme wind velocities, but from the dynamically changing wind directions and the impact of wind-borne debris. Similarly, any structure in the path of a hurricane may be simultaneously subjected to the severe forces of both wind and water, the greatest magnitude of each occurring at approximately the same time. The wind velocities in a hurricane may exceed design level winds and may subject the building to high winds first from one direction and then from nearly the opposite direction.

A4.3.6. Wind-induced structural damage can result from straight winds, downbursts, hurricanes, and tornadoes. The three primary damage mechanisms associated with severe windstorms involve:

- aerodynamic pressures created by flow of air around a structure (associated with all windstorms)
- pressures created by rapid atmospheric pressure fluctuations (associated primarily with tornadoes); and
- impact forces created by wind-borne missiles (associated with all windstorms).

A4.3.7. Examinations of structural damage caused by various types of windstorms, including tornadoes, suggest that most windstorm damage is caused by a combination of aerodynamic pressures and missile impacts. Atmospheric pressure fluctuations have little or no effect on the performance of ordinary structures because most ordinary structures have sufficient building envelope permeability (or venting) to allow equalization of pressures induced by atmospheric pressure changes. If the structure is airtight, as is the case with structures such as nuclear containment vessels, atmospheric pressure changes may significantly influence the performance of the structure.

A4.3.8. A basic understanding of the effects of wind pressures and missile impact forces assists the building evaluator. This, in addition to a general knowledge of the characteristics of the different types of structures, adds to the evaluator's ability to evaluate the life-safety of a building.

A4.4. Wind Pressures on Buildings. Wind pressures acting on buildings are distributed loads that are assumed to act normal to the building surface. Positive wind pressures act toward the surface of the building element and negative pressures act away from the building surface. The fundamental characteristics of wind pressures are described below based on the building component affected and the orientation of the building in the wind environment.

A4.4.1. Wind Pressures on Walls.

A4.4.1.1. Figure A4.1 presents a plan view of a simple rectangular building that is submerged in a wind flow as shown. Each wall of the structure is identified as a windward, side, or leeward wall depending upon its location with respect to the direction of wind flow. The windward wall is the wall facing the wind; the leeward wall is on the side opposite to the windward wall; and, the side walls are parallel to the wind flow.

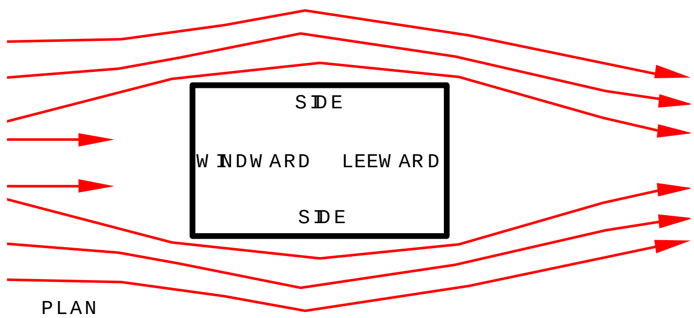


Figure A4.1. Wind Flow Around Simple Building

A4.4.1.2. Because the windward wall is perpendicular to the wind flow, the wind impinges directly on the windward wall producing positive pressures (Figure A4.2). As the wind flows around the windward corners, the local wind speed increases and the flow lines have a tendency to separate from the corner of the building. This causes the side walls to be subjected to negative pressures as shown. In addition, the turbulence and flow separations that occur at the windward corners of the building induce high negative pressures for short distances along the side walls. The leeward wall is also subjected to negative wind pressures that tend to be relatively uniformly distributed.

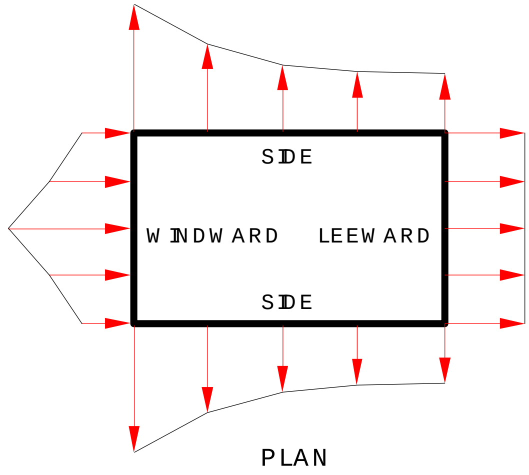


Figure A4.2. Wind Pressure on Walls

A4.4.2. Wind Pressures on Roofs.

A4.4.2.1. Most building roofs can be classified as flat roofs or gable roofs, depending on the shape of the roof and the direction of the wind with respect to the roof. Both types of roofs are discussed briefly in this section.

A4.4.2.2. Figure A4.3 presents a side view of a building with a flat roof. The wind is blowing from left to right. Figure A4.4 illustrates the wind pressures acting on the building. As stated previously, the windward wall is subjected to positive pressures and the leeward wall to negative pressures. As the wind flows upward and over the windward edge of the roof, the flow is accelerated and there is a tendency for the wind flow to separate from the roof. These flow characteristics result in the roof being subjected to negative pressures. In addition, because of the turbulence and flow separations around the windward roof corner, high negative pressures are generated for a short distance along the roof lines.

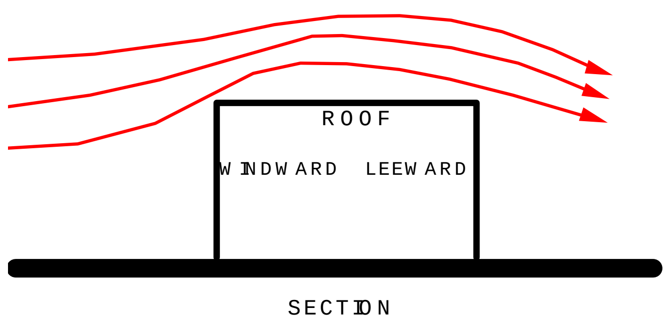


Figure A4.3. Wind Flow Over Simple Building with Flat Roof

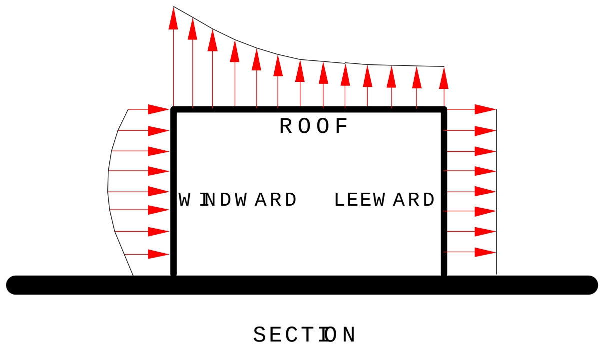


Figure A4.4. Wind Pressure on Flat Roof

A4.4.2.3. Figure A4.5 presents a side view of a building with a gable roof. The wind is blowing from the left to right. The character of the wind pressures on a gabled roof depends on the angle of the roof. Roofs with slopes less than 45 degrees are classified as low-sloped roofs and roofs with slopes that are greater than 45 degrees are classified as high-sloped roofs.

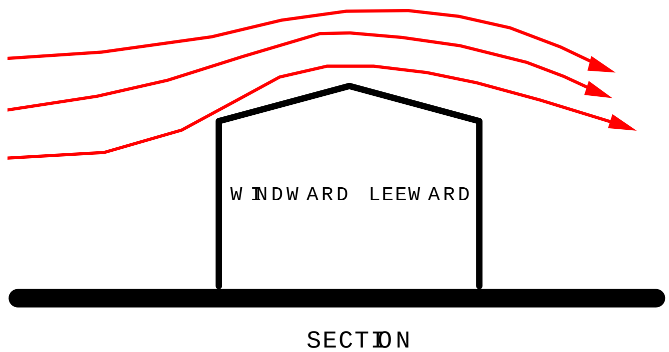


Figure A4.5. Wind Flow Over Simple Building with Gable Roof

A4.4.2.3.1. Figure A4.6 illustrates the distribution of pressures acting on a low-sloped roof with the wind flowing from left to right in a direction perpendicular to the roof ridge. As shown, both the windward slope and the leeward slope of the roof are subjected to negative pressures. In addition, locally high negative pressures can occur at both the windward eave or roof ridge. The magnitude of the local pressure excursions depends on the slope of the roof.

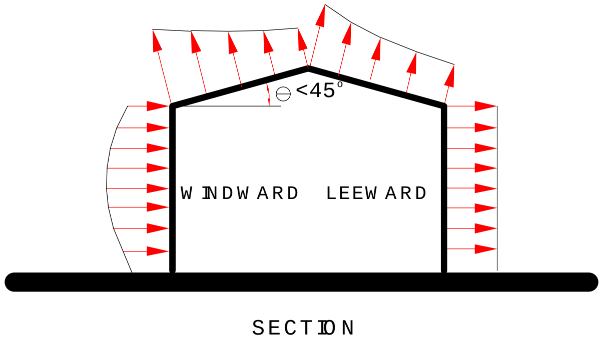


Figure A4.6. Wind Pressures on Low-Sloped Gable Roof

A4.4.2.3.2. Figure A4.7 shows the pressure distribution acting on a high-sloped roof with the wind blowing perpendicular to the roof ridge. The windward slope is subjected to either positive or negative pressures while the leeward slope is subjected to negative pressures. In addition, locally high pressure excursions are to be expected at the roof ridge. The magnitude of the ridge pressure fluctuations will depend on the slope of the roof.

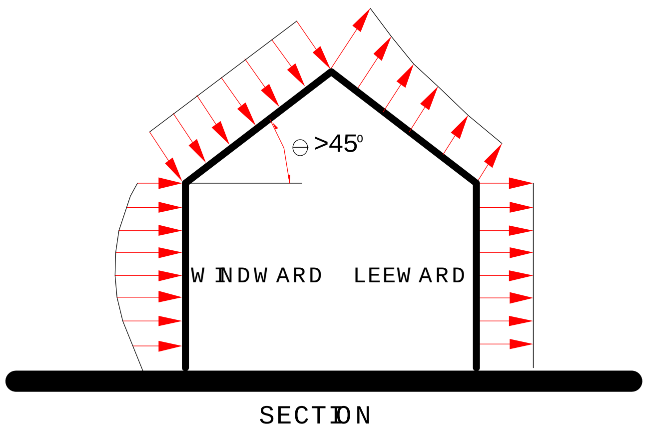


Figure A4.7. Wind Pressures on High-Sloped Gable Roof

A4.4.2.3.3. If the wind is flowing parallel to the roof ridge, the distribution of pressures on the roof is the same as for flat roofs.

A4.4.2.3.4. If either a gable or flat roof has an overhang, the roof will be subjected to high positive pressures on the windward overhang as depicted in Figure A4.8. If the overhang is associated with a flat roof or a low-sloped gabled roof, these forces will add to the overall roof uplift that must be resisted.

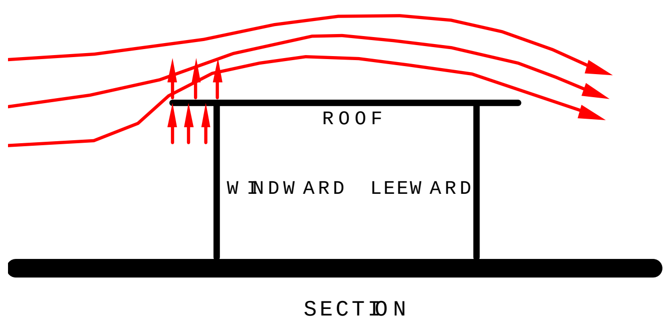


Figure A4.8. Wind Pressures Associated with Roof Overhang

A4.4.3. Internal Wind Pressures.

A4.4.3.1. If openings occur in the exterior building envelope during a windstorm, the internal building pressure is changed. The most common source of openings in a building during the windstorms is cladding that has failed (e.g., doors and windows). Missile impacts, as discussed in the following section, are a major cause of cladding failures. If the openings occur primarily on the windward wall (Figure A4.9), the internal pressure of the building will be increased and the walls and roof of the building will be forced outward. If the openings occur primarily on the side walls or the leeward wall

(Figure A4.10), the internal building pressure is reduced and the walls and roof of the building are pulled inward.

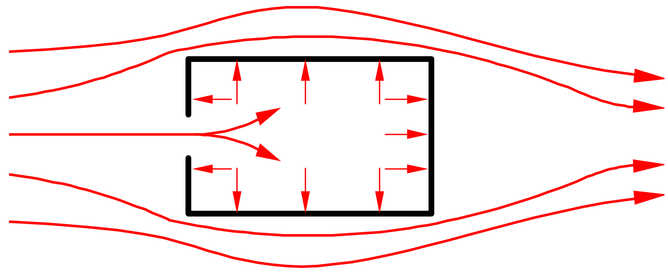


Figure A4.9. Internal Pressures Caused by Windward Wall Openings

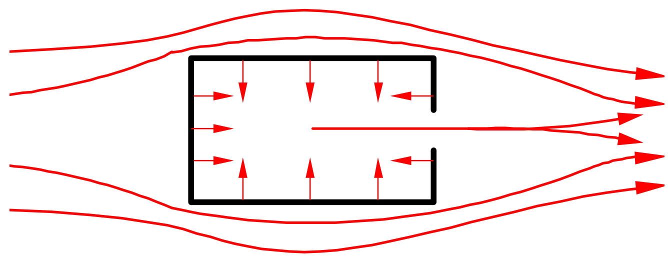


Figure A4.10. Internal Pressures Caused by Leeward Wall Openings

A4.4.3.2. The internal pressures add to the external pressures to cause resultant forces on the walls and roof surfaces of the building. The circumstances affecting a particular situation must be examined closely to determine the correct combination of pressures.

A4.5. Missile Impact Forces. In addition to the wind-induced pressures, structures located in the path of a severe windstorm are subject to impact forces caused by wind-borne debris. The character of the wind-borne missiles depends on the nature of the local construction practices. For current purposes, the population of wind-borne missiles can be divided into two groups: small missiles and large missiles.

A4.5.1. Small Wind-Borne Missiles.

A4.5.1.1. Small wind-borne missiles include objects such as roof gravel and small pieces of building fascia material. Small missiles are readily available in urban environments and are easily propelled even in relatively moderate straight-line winds. Further, small missiles are readily generated from the roofs of otherwise undamaged buildings. Once injected into the wind flow, small missiles such as roof gravel can be accelerated to velocities that approach the velocity of the wind. Small wind-borne missiles can be blown about in relatively moderate wind storms when measured wind speeds only marginally exceed 96.5 kilometers per hour (60 miles per hour).

A4.5.1.2. The primary effect of small missiles is damage to glass curtain walls. All types of glass, including annealed, heat-strengthened, tempered, laminated, and insulating glass can be broken by moderately-sized small missile impacts. When this occurs, the contents of the building are exposed to the effects of the windstorm. In addition, the openings introduced into the building envelope by the cladding failures permit the wind to enter the building causing internal pressure variations as discussed previously.

A4.5.2. Large Wind-Borne Missiles.

A4.5.2.1. If the wind storm is severe enough to initiate failure of surrounding structures, a wide variety of large missiles can be injected into the windfield and accelerated. These missiles include objects such as pieces of timber, sheet metal, and siding. Figure A4.11 illustrates a broad spectrum of missiles injected into the windstream when a marginally engineered structure at Homestead AFB failed during Hurricane Andrew.

Figure A4.11. Large Missiles Generated by Failure of Marginally Engineered Building

A4.5.2.2. If the intensity of the windstorm is sufficient, even larger missiles such as automobiles, aircraft, or partially intact roof assemblies can be accelerated in the wind field. Figure A4.12 shows an F-16 aircraft which was removed from the alert hangar at Homestead AFB by Hurricane Andrew. Other missiles such as metal roof decking, steel roof truss joists, hangar doors, and concrete blocks are also evident in the photograph. The large wind-borne missiles offer a formidable problem for the building designer. Rarely is it economically feasible to construct a structure that can withstand large missile impacts.

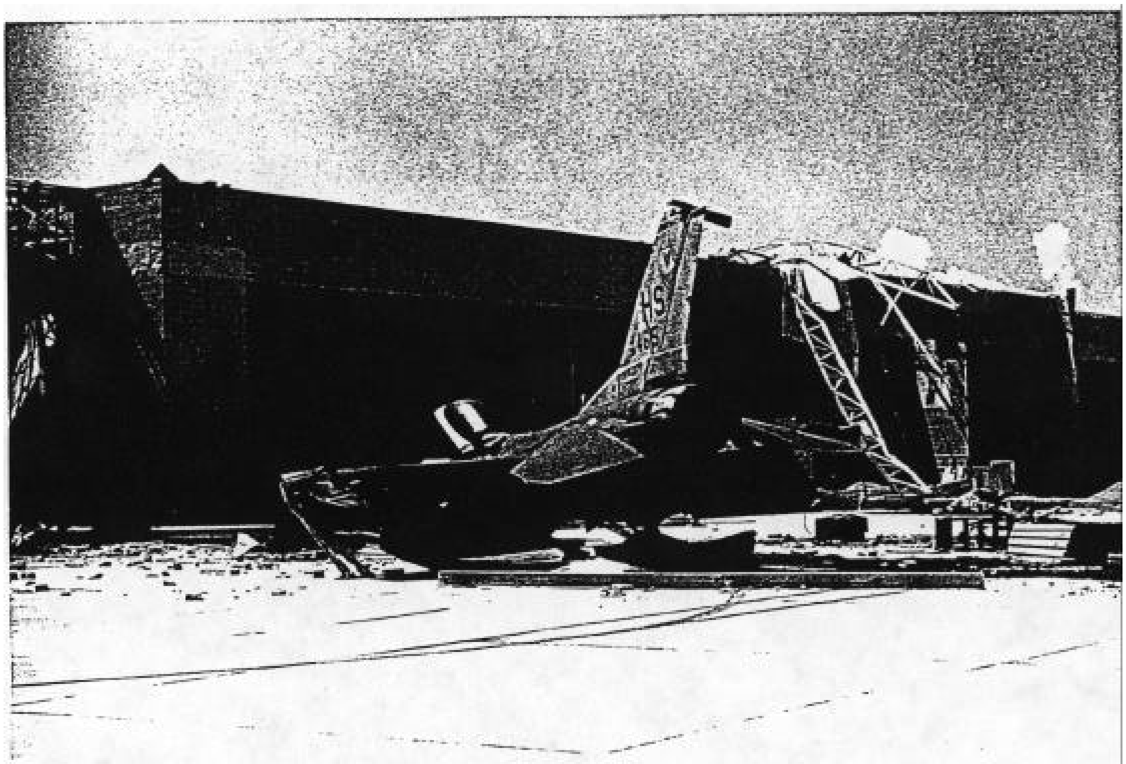


Figure A4.12. Large Missiles Generated by Failure of Fully Engineered Building

A4.6. Resistance of Buildings to Wind Loads.

A4.6.1. The design and construction of buildings to resist wind loads requires special attention to all construction and connection details. Also required is a change in thinking. It is natural to assume that a building must be supported to hold it up against the forces of gravity. For many people, it is not so natural to realize that the building must also be held down against the uplifting forces of wind on the roof, as well as restrained against lateral movement that would be caused by the pressures (and suction) on the walls. A building is an airfoil similar to an airplane or kite. As the wind passes over and around the building, the change in flow direction and velocity causes localized and general pressure changes that must be resisted by the structure as a whole and by the individual components of the structure; i.e., roof, walls, floors, foundations, doors, and windows, as well as the roofing and wall cladding. Figure A4.13 shows the general wind forces on a building and the principal resisting forces.

A4.6.2. Considering the transfer of wind forces into and through the building to the ground (the load path) will assist the evaluator in recognizing the effect of the wind on the building and in determining if the structure has the capacity to resist wind forces. Starting with the exterior, the wind forces are received by the building enclosure (roofing and wall cladding) or, as in the case of masonry construction, by the structure of the wall itself. The roofing must transfer these forces to the supporting roof deck or sheathing which must be attached to the roof structure (rafters, beams, and girders).

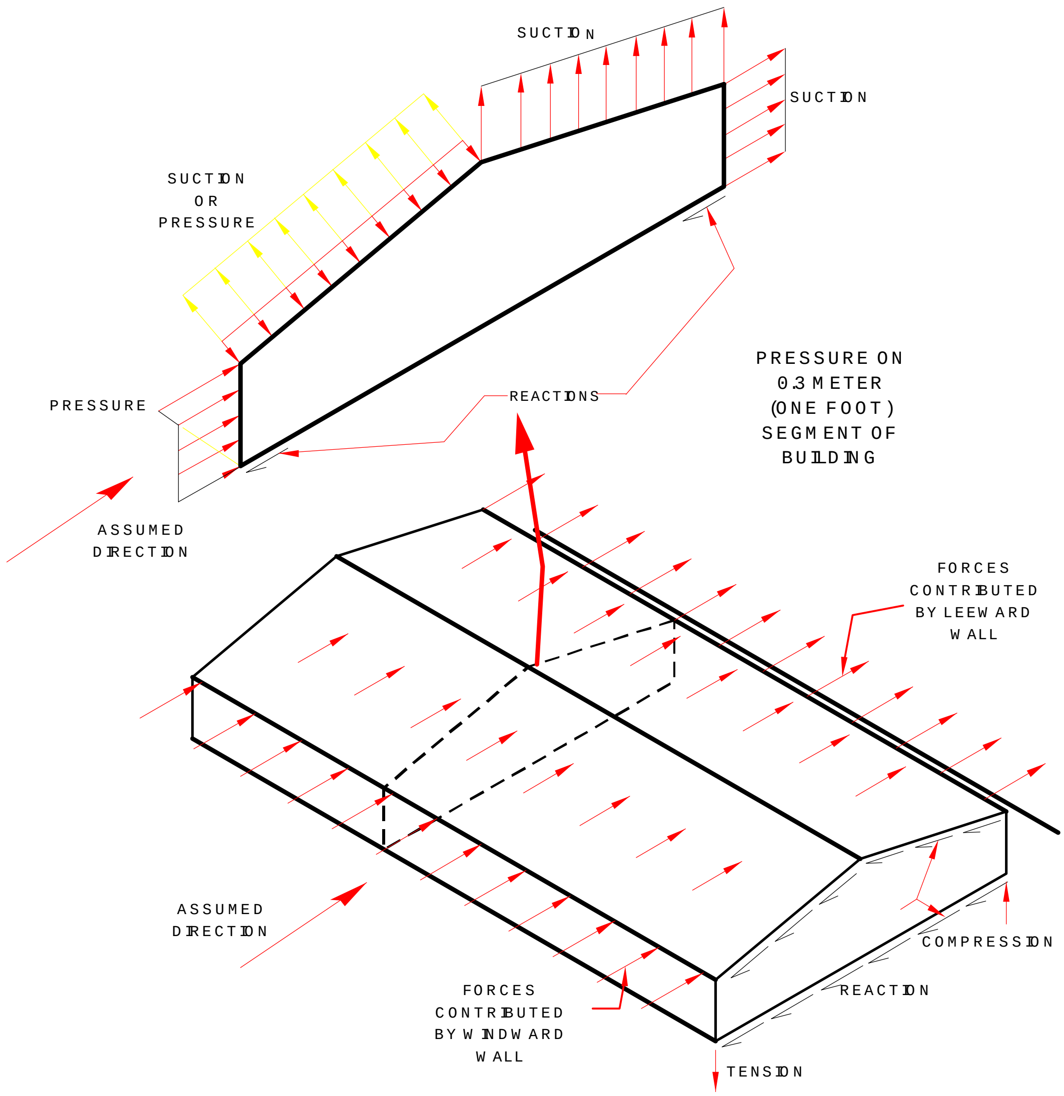


Figure A4.13. Wind Forces and Reactions

The wall cladding similarly must transfer these forces to wall structures (through the wall sheathing, if any) such as wall studs, steel or concrete framing, or concrete or masonry walls. The evaluator should consider the difference between cladding and structural walls. Brick veneer, for example, is cladding normally attached to wood frame structural walls with corrugated metal straps. However, a single wythe of brick can be constructed as the facing on a concrete block wall in such a manner that the brick and concrete block form an integrated structural bearing wall. Once the forces on the structure are transferred to the main building structure (the main wind force resisting system), the members of the building structure must be connected together so that the entire building is stable and acts as a system. The roof rafters, beams, and girders must be adequately connected to each other and to the walls or columns that support them; the walls or columns must be continuously connected until they reach the foundation to which they are connected; and the foundation must be capable of resisting the forces and transferring them to the ground. Similarly, each floor structure must be connected to the walls and columns. Additionally, the floor and roof structures are frequently used to provide lateral support to bearing and nonbearing walls at each level of the building. Thus, the connections between the various components of the building, structural and nonstructural, are most important.

A4.7. Building Types and Damage Implications. Buildings have been classified into four categories by wind researchers: fully engineered, pre-engineered, marginally engineered, and nonengineered. These four building categories are based on the degree of engineering attention afforded the building during the design phase. These building classifications are extremely useful in predicting the performance of a structure prior to a severe wind event. The building classifications are listed in order of decreasing survivability. Fully engineered buildings are the most survivable structures (least vulnerable), while nonengineered buildings are the least. It is rare for fully engineered structural frames to receive major damage even when exposed to the effects of tornadoes, and it is common for nonengineered structures to be severely damaged or destroyed in relatively mild windstorms. Therefore, a proper classification of building is valuable information for the building evaluator. The remainder of this section is devoted to discussions of these building classifications and examples of the expected performance of each type.

A4.7.1. Fully Engineered Buildings.

A4.7.1.1. Fully engineered buildings receive specific, individualized design attention from professional architects and engineers during the design and construction phases. Fully engineered buildings are equipped with redundant frame systems which are designed to resist the full effects of the wind and permit local overloads to be transferred to other portions of the structural system that are less stressed. The integrity of the main structural frame is not dependent on the survival of the cladding or secondary structural elements. Therefore, while the loss of cladding or secondary members of a fully engineered building usually results in severe water damage to the contents of the building, the primary building frame is rarely damaged by the effects of wind. Fully engineered building designs, by their nature are site-specific. Examples of fully engineered buildings include high-rise office and hotel buildings, hospitals, and public buildings.

A4.7.1.2. Figure A4.14 presents a photograph of a fully engineered aircraft maintenance hangar exposed to the effects of Hurricane Andrew at Homestead AFB, Florida. In this case, the roof decking, metal siding, and hangar doors sustained significant damage, resulting in damage to the contents and debris throughout the hangar. It is significant that the hangar door panels failed to remain supported, most probably due to upward deflection of the roof trusses caused by wind uplift, possibly aggravated by deflection of the door itself. The internal pressure that resulted upon loss of the doors probably contributed to the cladding failure. This is typical of the type of damage that fully engineered buildings sustain in severe windstorms. Only in the most severe situations will fully engineered buildings experience major structural damage.

Figure A4.14. Fully Engineered Hangar with Nonstructural Damage to Cladding

A4.7.2. Pre-Engineered Buildings.

A4.7.2.1. Pre-engineered buildings are usually marketed in similar units across broad areas of the country. The basic building system generally receives significant engineering attention prior to the manufacturing process. Pre-engineered buildings are usually designed to resist the full effects of wind forces. However, unlike fully engineered buildings, the main structural frame possesses little redundancy, and the individual frame members often depend on the cladding and secondary members for lateral support. This design philosophy results in a structure whose performance is governed by “weakest link” behavior. If a single element in the structural system fails, a progressive or domino type failure will be initiated. The problem is exacerbated by the fact that there are variations in individual constructions such as the number and size of the openings, doors, and windows. Further, the placement and wind resistance of the doors and windows is generally not reviewed by the original design team, thus introducing building components that are weaker than the rest of the structure. The failure of these weak components compromises the integrity of the entire building. Examples of pre-engineered construction include metal buildings and manufactured housing units. It is noteworthy that over 50 percent of the nonresidential construction in the United States falls into the category of pre-engineered metal buildings.

A4.7.2.2. Figure A4.15 shows a pre-engineered metal building at Homestead AFB that failed as the result of Hurricane Andrew. In this case, the roof purlins spanning between the windward wall and the primary structural members buckled due to the combined effects of axial compression created by the forces exerted from the windward wall and bending caused by the local uplift effects on the roof. Once the began, it spread throughout the structure. This effect is depicted in Figure A4.16.

Figure A4.15. Failure of End Wall of Pre-Engineered Metal Building

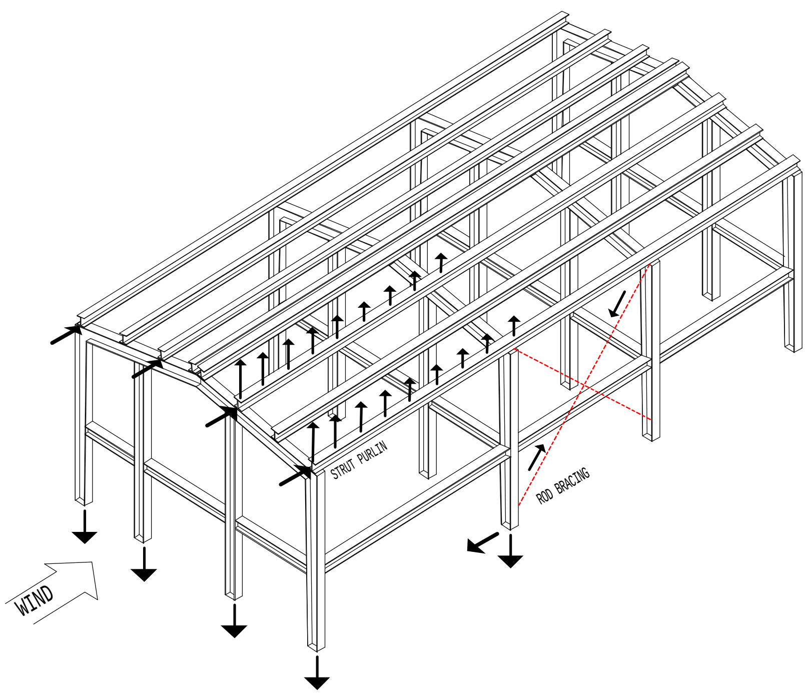


Figure A4.16. Forces on Pre-Engineering Metal Building

A4.7.3. Marginally Engineered Buildings.

A4.7.3.1. The design and construction of marginally engineered buildings is usually accomplished by a combination of local construction practices with only minimal engineering input. These buildings are usually built with some combination of masonry, light steel framing, open-web steel joists, wood framing, and wood rafters. The walls and roof typically are designed and constructed as independent units with primary consideration given to gravity loads. Rarely are walls and roofs built to resist uplift or lateral pressures caused by wind, and almost no thought is given to maintaining structural continuity throughout the building system. Marginally engineered structures tend to fail primarily at connections. Failure of the roof-to-wall, or wall-to-foundation connections usually initiate the failure of marginally engineered structures. It is not uncommon for entire roof or wall assemblies to be separated from a marginally engineered structure and blown intact across significant distances. Examples of marginally engineered structures may include motels, and commercial and light industrial buildings.

A4.7.3.2. Figure A4.17 shows the failure of a marginally engineered building at Homestead AFB subjected to Hurricane Andrew. This building lost a major portion of its roof and windward wall, as well as the sidewall shown in the photograph. In this case, the 12-inch concrete block was unreinforced except for joint reinforcing at 16-inch intervals and a bond beam at the top of the wall. Without vertical reinforcing to tie the block to the bond beam, the combination of pressure on the wall and uplift left the bond beam suspended in air after the wall had failed.

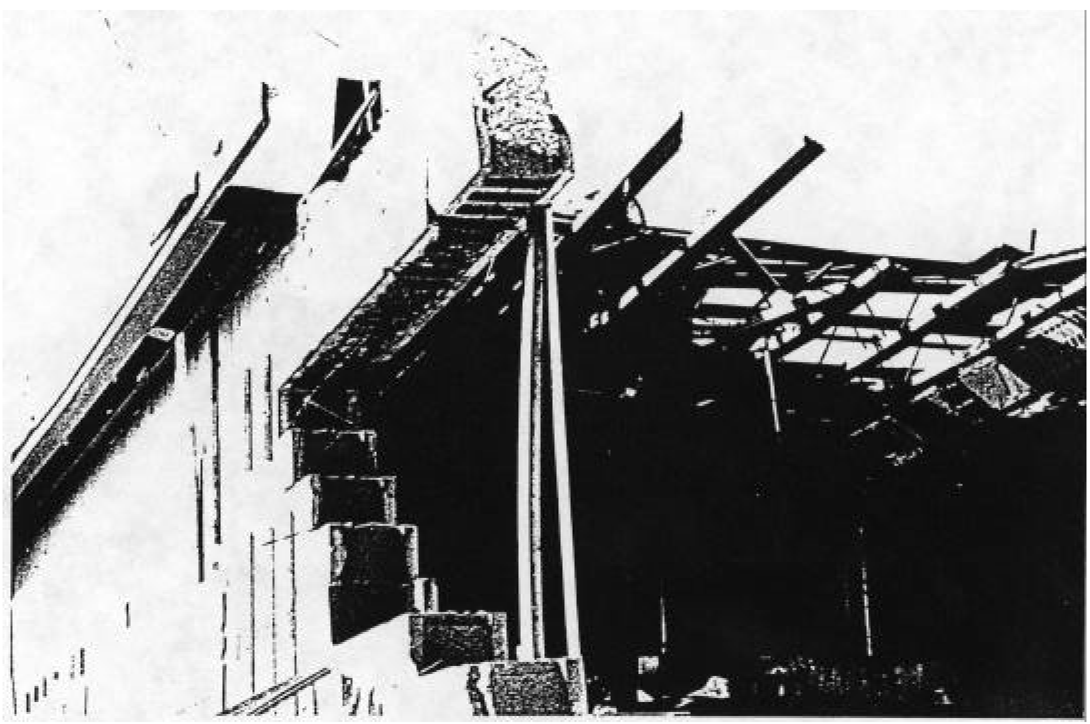


Figure A4.17. Failure of a Marginally Engineered Building

A4.7.3.3. The above cases demonstrate the general characteristics of the failure of marginally engineered structures. It should be noted that when evaluating the structures, lack of knowledge or attention to details during construction has more to do with the severity of the potential damage than the effects of the wind.

A4.7.4. Nonengineered Buildings. The design and construction practice associated with nonengineered buildings are completely dominated by local custom and local prescriptive building codes. These structures receive little specific engineering attention. Wind-related failures associated with nonengineered buildings range from failures initiated by inadequate component strength to failures initiated by poor connections. Examples of nonengineered buildings include most single and multifamily residences, most one- and two-story apartment units, and some small commercial buildings.

A4.7.5. Summary.

A4.7.5.1. In general, the amount of engineering in the design has a pronounced effect on the performance of a building when subjected to any disaster. Fully engineered structures (i.e., buildings that are individually designed and usually contain redundant structural systems) will, in most cases, withstand wind forces. Pre-engineered structures are usually designed to require intact exterior cladding to resist the design forces. Since damage to cladding is a typical result of wind forces, pre-engineered buildings are somewhat more susceptible to wind damage; however, marginally engineered and nonengineered buildings that are constructed more by acceptable practice than engineered design are highly susceptible to wind damage. These buildings are typically constructed of unreinforced masonry and/or wood. Unreinforced masonry foundation walls in particular are responsible for a large amount of wind damage because of the low factor of safety against lateral-load application. Major damage to wood frame structures is also common as a result of inadequate connections to maintain structural integrity. This does not rule out damage to heavier types of engineered construction. Any structure in the path of windborne missiles can be damaged.

A4.7.5.2. It is important to note, however, that when one structural element is damaged or fails, the integrity of the entire system may be at risk. In general, all the structural components of a building are designed to work together to resist the loads applied to the structure. One weak or missing link in the system can result in reduced capacity or damage to other components and perhaps collapse of the building, especially in marginally engineered and nonengineered structures.

A4.8. Wind Loads on Structures for Rapid and Detailed Evaluation.

A4.8.1. Simplifying Assumptions. For buildings less than 12.2 meters (40 feet) high, use z = h (mean roof height or eave height for roof slope of less than 10 degrees). Therefore, $q_z = q_h$ and $G_z = G_h$ will simplify force calculations by using a constant pressure with height on the windward wall. This assumption is conservative, as it results in pressures up to approximately 23 percent higher than required on the lower portion of the windward wall (0-4.6 meters [0-15 feet]). Pressures on the leeward wall, side walls, and roof are not affected. This assumption will result in overall building forces on the Main Wind Force Resisting System (MWFRS) less than 10 percent over those calculated by more accurately considering the variation of pressure with height on the windward wall.

A4.8.2. Complete the Wind Lateral Load Calculation Data form (Attachment 6) using the following procedures with reference to ASCE 7-95 (see also Reference 17 for guidance):

- Determine basic wind speed, V (paragraph 6.5.2 and Figure 6.1).
- Determine importance factor, I (paragagraph 6.5.1, Table 1.1, and Table 6.2).
- Determine exposure category (paragraph 6.5.3). Most buildings on Air Force bases should be considered to be in Exposure C or higher due to the open terrain of the airfield.
- Determine velocity pressure exposure coefficient, K_z (paragraph 6.5.1 and Table 6.3) and $K_{zt} \geq 1.0$ where applicable (paragraph 6.5.5).
- Calculate the velocity pressure, $q_z = 0.00256 K_z K_{zt} V^2 I$ (paragraph 6.5.1 and Equation 6.1).
- Determine gust effect factor, G; or, for flexible structures, G_f (paragraph 6.6).
- Determine MWFRS external pressure coefficients, C_p (paragraph 6.7.1, 6.7.2, and Figure 6.3).
- Determine internal pressure coefficients, GC_{pi} (paragraph 6.7.1 and Table 6.4). Consider the potential strength of doors, windows, and siding when determining if breaching of the building envelope could create an opening which would cause a building to be evaluated under Condition II.
- Calculate design wind pressures, p, and forces, F (paragraph 6.4.2 and Table 6.1).

$$p = qGC_p - q_h(GC_{pi}) \qquad \text{(A4-1)}$$

Consider all notes at the bottom of Table 6.1. Combine external and internal pressures to ascertain the most critical load.

A4.9. Procedures for Rapid Evaluation of Existing Buildings for Wind Loads.

A4.9.1. Compare wind pressures and forces to seismic loads and forces to evaluate governing loads. Note that wind may govern the design of some members while seismic may govern design of others. Similarly, wind may govern design in one direction (transverse), while seismic may govern in the other direction (longitudinal).

A4.9.2. Complete the wind evaluation statements and review seismic evaluation statements as necessary to address appropriate considerations. Wind evaluation statements are supplemental to seismic evaluation statements. For buildings exempt from seismic evaluation, seismic evaluation statements shall be used to assure complete coverage of relevant issues.

A4.9.3. Perform appropriate "Quick Checks" as in FEMA 178 using the demand calculated above.

A4.9.4. Compare demand with capacity determined using FEMA 178 procedures to determine capacity; i.e., multiply working stresses by factors given for the various materials (1.7 for steel). Note that this evaluation technique is significantly unconservative for wind loads.

A4.9.5. Components and cladding (nonstructural components) should be checked for performance when subjected to wind loads. The procedures for nonstructural evaluation of building components are provided in other publications.

RAPID SEISMIC AND WIND STRUCTURAL EVALUATION

A5.1. Introduction. The rapid evaluation procedure will quickly identify those buildings which have a complete lateral-force-resisting system of a minimum required strength. A registered professional engineer experienced in seismic and wind design shall do the necessary work of this procedure.

A5.1.1. The preliminary seismic structural evaluation is based on *NEHRP Handbook for the Seismic Evaluation of Existing Buildings,* FEMA 178, June 1992 (Reference 4). The format is the same as contained in the U. S. Postal Service *Procedures for Seismic Evaluation of Existing Buildings*, ATC-26-1 (Reference 19). Formulas and procedures are reproduced from FEMA 178 herein, but the engineer should refer to FEMA 178 for the necessary background and supplemental information.

A5.2. Exemptions. Those buildings which are exempt benchmark buildings and are Immediate Occupancy or High Risk (Category I or III) located in regions with the National Design Force Exceedance Factor (NDFEF) greater than 1.5 shall be returned to the list of buildings within the seismic risk inventory designated for structural evaluation.

A5.3. Rapid Structural Evaluation Procedure. The rapid structural evaluation procedure has three basic steps: visit the site and collect data, determine the building type and review the evaluation statements, and perform the followup field work. The following paragraphs summarize the procedure and provide supplementary information.

A5.3.1. Preliminary Work.

A5.3.1.1. Visit the office of the Project Engineer and obtain record drawings and available background information.

A5.3.1.2. Make a tentative identification of the building type using Attachment 10.

A5.3.1.3. Make a tentative review of the evaluation statement (FEMA 178, Appendixes A, B, and G).

A5.3.1.4. Plan the site visit with the assistance of the Project Officer. A large part of the work of the rapid structural evaluation is determining the building type and the condition of the building. The amount of work involved in this depends primarily on the availability of drawings and accessibility of components; it is not directly related to the size of the building or the building type. Buildings can be characterized as having one of the following general levels of accessibility.

A5.3.1.4.1. Easy Access. This may be a simple, unfinished pre-engineered building, or a building that is not so simple but is open on the interior and has few interior walls.

A5.3.1.4.2. Moderately Easy Access. This is a building or a portion of a building that offers access behind or beneath the finishes in attics, plenums, and crawl spaces.

A5.3.1.4.3. Difficult Access. This is a building or a portion of a building that has extensive wall finishes, hard ceilings, and exterior veneers that conceal the structural components.

A5.3.2. Work at the Site.

A5.3.2.1. Review the drawings that are available.

A5.3.2.2. If no drawings are available, obtain the following information:

- overall plan dimensions
- bay sizes
- number of stories
- story heights
- descriptions of building systems for calculation of weights for seismic analysis:
  - roof
  - floor
  - exterior walls
  - interior walls and partitions

A5.3.2.3. Walk around the perimeter, through the building, and on the roof, noting general conditions.

A5.3.2.4. Take photographs to document general views of the building, general structural framing that may be exposed, and additional specific details that the evaluator believes are important. Permission will be required prior to taking photographs on the installation.

A5.3.2.5. Make detailed observations of the building type and the building condition.

A5.3.2.5.1. Determine material conditions, looking for rust, wood decay, cracking, sagging, or other signs of deterioration or distress. Material strengths should be determined by available information or by the best judgment of the evaluating engineer. If it is believed that more detailed examination or testing is necessary to establish sufficiently accurate assessment of material condition or strengths, the evaluating engineer should include appropriate recommendations in the rapid structural evaluation executive summary on the Evaluation Results form describing the approximate costs of such assessment, and the nature and significance of compromise in evaluation accuracy should such assessment not be made. In any event, the rapid structural evaluation will be completed based upon a judgment of material conditions and strengths by the evaluator formed without the benefit of material testing or more detailed examination. If conducted, material testing and more detailed evaluation will be done as part of the detailed structural evaluation.

A5.3.2.5.2. Determine the building type, looking at components, and anchorage of components to the foundations (Attachment 10).

A5.3.2.5.3. Note the location and approximate weight of any heavy building mechanical equipment.

A5.3.3. Followup Work.

A5.3.3.1. Assemble and review available drawings, soil reports, reports of previous investigations, and any other existing material. If the building was designed to a seismic code, there will probably be an indication of this among the general notes on drawings concerning design loads. The Project Engineer may have useful information that is missing from the plans: the dates of construction, repairs, and remodels.

A5.3.3.2. Complete evaluation data forms provided in Attachment 6*, Evaluation Data Collection*.

A5.3.3.3. Address the evaluation statements for wind and seismic.

A5.3.3.3.1. Quick Check Statements. See paragraph A5.3.5 below.

A5.3.3.3.2. Other Statements. It is expected that calculations will be limited to the quick check procedures. When quick check procedures and simple calculations will not resolve concern about a perceived deficiency, the evaluation statement should be considered false. The justification for the false statement and the recommendation for a detailed structural evaluation shall be included in the report (Attachment 8).

A5.3.3.3.3. Configuration Issues. The evaluating engineer should use some judgment in dealing with the configuration statements. Some configuration conditions are obvious; others are marginal. The marginal issues may require substantial calculations to determine whether NEHRP-defined irregularities exist. If an issue cannot be quickly resolved, it shall be deferred to a detailed structural evaluation. An extensive analysis will not be conducted during the rapid structural evaluation.

A5.3.3.3.4. Peak Velocity-Related Acceleration Coefficient, A_V. Some evaluation statements require the use of A_V. In these cases, determine the appropriate value of A_V as being equal to $0.667S_{DL}$ as determined in Attachment 2.

A5.3.3.4. From the evaluation statements that are found to be “False,” make a list of the deficiencies that identify evaluation concerns. Refer to FEMA 178, Chapters 3 through 10 for guidance.

A5.3.3.5. It may be necessary to make a followup visit to the building to confirm information, answer questions that come up during the later stages of the evaluation, and examine conditions revealed by previously arranged removal of finishes.

A5.3.3.6. Write the Report. See paragraph A5.3.6.

A5.3.4. Identification of Building Type Procedure. The following procedure is recommended for identifying the building type.

A5.3.4.1. Identify the major components of the lateral-force-resisting system; i.e., diaphragms, shear walls, braced frames, and moment frames.

A5.3.4.2. Identify the connections between the components of the lateral-force-resisting system, particularly the connections between the horizontal diaphragms and the vertical components.

A5.3.4.3. Identify the building type according to the following list. Refer to Attachment 10 for definitions and guidance in identifying the building type.

1. Wood: residential
2. Wood: commercial and industrial
3. Steel moment frame
4. Steel braced frame
5. Steel light frame
6. Steel frame with concrete shear walls
7. Steel frame with masonry walls
8. Concrete moment frame
9. Concrete shear walls
10. Concrete frame with masonry walls
11. Precast concrete tilt-up walls
12. Precast concrete frames with concrete shear walls
13. Reinforced masonry bearing walls with wood or metal deck diaphragms
14. Reinforced masonry bearing walls with precast concrete diaphragms
15. Unreinforced masonry bearing wall buildings

A5.3.5. Quick Checks of Strength and Stiffness. Before embarking on a conventional analysis of a building, the evaluating engineer is encouraged to make a quick check, a "back-of-the-envelope" estimate, of the strength and stiffness of the building. For most of the fifteen common building types, a "Quick Check" procedure is specified. As described in paragraph 5.4.4 of this ETL, a two-level procedure is used in conducting quick check assessments. Level A calculations are conducted using the "design earthquake." Should the NDFEF exceed 1.5, a supplemental check using 2/3 times the spectral acceleration of the Maximum Considered Earthquake is conducted to ensure structural adequacy of the building against collapse should the large earthquake occur. The same quick check formulas and procedures are used in conducting Level A and Level B rapid evaluations. Only the force levels differ. In the review of an existing structure, it may be necessary to check the average shear stress or drive for upper stories in addition to the first story. In this case, the story shear for an upper story may be approximated as follows.

A5.3.5.1. Seismic Quick Check Story Shears.

$$V_j = \left(\frac{n+j}{n+1}\right)\left(\frac{W_j}{W}\right)1.2V \quad \textbf{Error! Switch argument not specified.} \qquad \text{(A5-1)}$$

where:

V_j = maximum story shear at story Level j

n = total number of stories above ground level

j = number of story levels under consideration

W_j = total seismic dead load of all stories above Level j

W = total seismic dead load

V = base shear from Attachment 3 for earthquakes and Attachment 4 for wind

A5.3.5.2. Wind Quick Check Story Shears. Determine the approximate wind pressures and forces on the structure in accordance with Attachment 4 and ASCE 7-95 (Reference 3). Calculate the story shears using principles of structural mechanics. Increase the calculated quantities by a factor of 1.2 in consideration of the approximate nature of the quick check formulas.

A5.3.5.3. Quick Check Details. "Quick Checks," where appropriate, are triggered by evaluation statements in the lists in FEMA 178, Appendixes A and B (Reference 4). The details for particular checks follow.

A5.3.5.3.1. Story Drift for Moment Frames. The drift ratio is based on the deflection due to flexural displacement of a representative column, including the effect of end rotation due to bending of the representative girder. The following equation for the drift ratio is applicable only to regular, multistory, multi-bay frames with columns continuous top and bottom.

$$DR = \left(\frac{k_b + k_c}{k_b \cdot k_c}\right)\left(\frac{h}{12E}\right)V_c C_d \qquad \text{(A5-2)}$$

where:

DR = drift ratio = interstory displacement divided by interstory height

k_b = I/L for the beam

k_c = I/h for the column

h = story height (m [in])

I = moment of inertia (m^4 [in^4])

L = center to center length (m [in])

E = modulus of elasticity (Pa [ksi])

V_c = shear in the column (N [kips])

C_d = deflection amplification factor, Table A3.5, Attachment 3

A5.3.5.3.1.1. Equation A5-2 also is appropriate for reinforced concrete frames if appropriate cracked section properties are used. For other configurations of frames, compute the drift ratio from the principles of structural mechanics.

A5.3.5.3.1.2. Drift ratios for seismic evaluation shall not exceed the limits provided in Table A8.1, Drift Ratio Limits, Attachment 8. Drift ratios for wind evaluation shall not exceed 0.002, 0.004, and 0.005 for Category I, III, and IV buildings, respectively.

A5.3.5.3.2. 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:

$$v_{avg} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \tag{A5-3}$$

where:

n_c = total number of columns

n_f = total number of frames in the direction of loading

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

V_j = story shear from Equation A5-1

Equation A5-3 assumes that nearly all of the columns in the frame have similar stiffness.

A5.3.5.3.3. Shearing Stress in Shear Walls. The equation for a quick estimate of the average wall shear stress, (v_{avg}), is as follows:

$$v_{avg} = \frac{V_j}{A_w} \tag{A5-4}$$

where:

Vj = story shear at the level under consideration determined from Equation A5-1

A_w = summation of the horizontal cross sectional area of all shear walls in the direction of loading. The wall area should 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.

A5.3.5.3.3.1. The allowable stresses for the various types of shear walls are given in the commentary on the various evaluation statements; FEMA 178, Sec. 5.1 (Reference 4) for concrete shear walls; FEMA 178, Sec. 5.3 (Reference 4) for reinforced masonry shear walls; FEMA 178, Sec. 5.4 (Reference 4) for unreinforced masonry shear walls; and FEMA 178, Sec. 5.6 (Reference 4) for wood shear walls.

A5.3.5.3.4. Diagonal Bracing. The equation for a quick estimate of the average axial stress in the diagonal bracing, (f_{br}), is as follows.

$$f_{br} = \left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \qquad \text{(A5-5)}$$

where:

L_{br} = average length of the braces (m [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 (m [ft])

A_{br} = the average area of a diagonal brace (m^2 [in^2])

V_j = maximum story shear at each level (N [kips])

A5.3.6. Report.

A5.3.6.1. Report the results of the preliminary evaluation by completing the entire rapid structural evaluation executive summary form and that portion of the Evaluation Results form dealing with the rapid structural evaluation. Both of these forms are provided in Attachment 6, *Evaluation Data Collection*.

A5.3.6.1.1. Evaluation Results Form. The deficiencies identified during the wind and seismic evaluations will be listed on the evaluation results form. Completed evaluation statements, supporting calculations, and photographs with descriptions will be attached to the evaluation results form.

A5.3.6.1.2. Rapid Structural Evaluation Executive Summary. A rapid structural evaluation executive summary will be completed, including a summary of wind and seismic vulnerability, an engineering recommendation for mitigation, and the evaluating engineer's level of confidence in the results of the rapid structural evaluation. The deficiencies identified during the wind and seismic evaluations will be listed on the evaluation results form.

A5.3.6.2. The report will be based on the following considerations: Except where the need for retrofit is obvious or the building is judged to be satisfactory by the rapid structural evaluation (passes Level A and B), the building will be recommended for seismic detailed structural evaluation.

A5.3.6.2.1. The building is acceptable. No further evaluation is needed if all evaluation statements are found to be "True;" the Level A quick checks are satisfied; and, if the NDFEF is greater than 1.5, the Level B quick checks are conducted and found to be satisfactory.

A5.3.6.2.2. The building needs minor fixing. The building deficiencies are simple and clear and can readily be repaired at moderate costs. The evaluator will recommend on the rapid structural evaluation executive summary form that the building proceed to the development of upgrade scheme and related costs.

A5.3.6.2.3. The building has marginal capacity. A detailed structural evaluation is required to determine whether remedial work is needed, and, if needed, the extent of work required. The evaluator will recommend the building for continued evaluation on the rapid structural evaluation executive summary form.

A5.3.6.2.3. The building needs major repair work. The building deficiencies are so severe that the building is in need of major work. The evaluator will recommend on the rapid structural evaluation summary form that the building be demolished or be processed for the development of upgrade schemes and related costs.

A5.3.7. Hours Required for Rapid Structural Evaluation. Table A5.1 presents estimates of time required for preliminary evaluation of low, high, and special complex buildings. The low estimate of 20 hours assumes that the building is simple, has accessible components, is conveniently located, and the drawings are available (Evaluation Example, Attachment 18). The high estimate of 50 hours, which is not necessarily a maximum, assumes that the building is more complex, has less accessible components, is farther away, but drawings are available (Evaluation Example, Attachment 13). Special buildings are unusually complex, irregular configurations, have critical inaccessible components, and may or may not have drawings available (Evaluation Example, Attachment 13). The time required for special buildings should be estimated on a case-by-case basis using Table A5.1 as a guide. These time estimates assume most of the work is done by an experienced senior engineer. Engineers with limited experience will need more time.

Table A5.1. Representative Time Estimates for a Range of Rapid Evaluation (Hours)

	Special	High	Low
Preliminary Work			

Mobilization	4.0	1.0	1.0
Obtain and review drawings	5.0	3.0	1.0
Identify building type	1.0	1.0	Type is obvious
Plan the site visit	2.0	1.0	Building is simple
Work at the Site			
Travel	5.0	2.0	1.0
Obtain building data	4.0	2.0	1.0
Walk through	8.0	2.0	0.5
Examine Components**	16.0	4.0	0.5
Followup Work			
Analyze building data	5.0	2.0	1.0
Calculate lateral forces*	6.0	2.0	1.0
Address statements*	9.0	6.0	3.0
Perform quick checks*	3.0	2.0	1.0
List deficiencies	2.0	1.0	1.0
Return to site	8.0	4.0	Not needed
Develop retrofit scheme/cost estimate	24.0	8.0	4.0
Develop conclusions and write report	16.0	8.0	4.0
Answer questions	2.0	1.0	No questions
Total Professional Hours	120.0	50.0	20.0

*Time estimates apply to wind or seismic evaluation separately. If both apply, double time for these items.

**May require rental of equipment.

EVALUATION DATA FORM

BUILDING AND SITE DATA

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use ____

Seismic Risk Code (W/XXXX/YY YY/ZZZ Z) __________________________________

Evaluator ________________________________ Date ___________________________

Document Availability:

- Construction Drawings ________________
- Specifications ___________________________
- Shop Drawings __________________________
- Structural Design Analysis

Site and Soil Parameters:

- Seismicity ________________
- Geotechnical Report _________
- Soil Profile Type ____________

Building Data:

Length ______________ Width ____________ Height ____________ Stories ______

Description of Structure: ____

____

____

____

General Condition:

- Visible General Deterioration ____
- Specific Deterioration of Structural Systems
  - by alterations or removal ____
  - by prior earthquake ____
  - by prior wind ____
  - by prior fire ____
  - by weathering ____
  - by corrosion ____
  - other ____

EVALUATION DATA FORM

STRUCTURAL DATA

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use _______________________________________

Lateral--Force-Resisting System:

Diaphragms (Describe briefly)

	Type	Connection to Vertical Elements
Roof	___________________________	____________________________________
Floor	___________________________	____________________________________
Floor	___________________________	____________________________________

Vertical Lateral Force Resisting Elements:

Shear wall ____

Vertical bracing ____

Rigid frames ____

Infill frames ____

Unusual Features (Describe briefly):

Plan irregularity ____

Vertical Irregularity ____

Diaphragm discontinuity ____

Sloping building site ____

Other ____

EVALUATION DATA FORM

STRUCTURAL DATA

Base ______________ City ______________ County __________ State __________

Building ______________ Building Use ____

Structural Conditions:

Evidence of foundation settlement ____

Building variations from construction drawings

Additions to building/date ____

Configuration compatibility to additions ________________________________

Specific structural modifications ____

Gravity Load Resisting System:

Structural Features

Roof Framing ____

Floor Framing ____

Ground Floor ____

Basement ____

Exterior Walls ____

Openings ____

Number ______________ Large ______________ Small ______________

Columns: _

Foundations:

Spread footings ____

Strip footings ____

Pier footings ____

Piles ____

Caissons ____

EVALUATION DATA FORM

SEISMIC LATERAL LOAD CALCULATION DATA

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use ______________________________________

Site Parameters:	**EQ-I**	**EQ-II**	**EQ-III**
Return Period	___________	___________	___________
Spectral Acceleration, Short Period	___________	___________	___________
Spectral Acceleration, Long Period	___________	___________	___________

NDFEF (Table A2.1) ______________________

Soil Profile Type (Table A3.2) ______________________

Site Coefficient, Fv (Table A3.3) ______________________

Site Coefficient, Fa (Table A3.4) ______________________

Building Parameters:	**Transverse**	**Longitudinal**
Building Type (Attachment 10)	____________	____________
Response Modification Coefficient, R (Table A3.6)	____________	____________
Building Period, T (Section A3.2.6.1)	____________	____________

Rapid Seismic Evaluation:

Level A (EQ-II using R)

Seismic Design Coefficient, Cs (A3.2.3.1)	____________	____________
Limiting Seismic Design Coefficient, Cs (A3.2.3.1)	__________	____________
Seismic Base Shear, V=CsW	____________	____________

Level B (2/3 EQ-III using R)

Seismic Design Coefficient, Cs (A3.2.3.2)	____________	____________
Limiting Seismic Design Coefficient, Cs (A3.2.3.2)	_	_
Seismic Base Shear, V=CsW	____________	____________

EVALUATION DATA FORM

WIND LATERAL LOAD CALCULATION DATA

Base ______________ City ______________ County ___________State ___________

Building ______________ Building Use _______________________________________

Wind Load Parameters (Ref. ASCE 7-95):

Basic wind speed, V ________________________

Importance factor, I ________________________

Exposure category ________________________

Exposure coefficient, Kh ________________________

Topographic factor, Kt ________________________

Velocity pressure, qz ________________________

Internal pressure coefficient, GCpi ________________________

EVALUATION DATA FORM

RAPID STRUCTURAL EVALUATION RESULTS FORM

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use ___________________________________

Seismic Risk Code (W/XXXX/YY YY/ZZZ Z) ___________________________________

Evaluator ________________________________ Date ___________________________

Wind Evaluation

Identified Deficiencies:

1. ______________________________
2. ______________________________
3. ______________________________
4. ______________________________
5. ______________________________
6. ______________________________
7. ______________________________
8. ______________________________

Seismic Evaluation

Identified Deficiencies:

1. ______________________________
2. ______________________________
3. ______________________________
4. ______________________________
5. ______________________________
6. ______________________________
7. ______________________________
8. ______________________________

Schematic Retrofit Concept

Programming Structural Cost Estimate

Total Program Cost Estimate

Engineer’s Summary Statement:

EVALUATION DATA FORM

RAPID NONSTRUCTURAL EVALUATION RESULTS FORM

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use ____

Seismic Risk Code (W/XXXX/YY YY/ZZZ Z) ____________________________________

Evaluator ________________________________ Date ___________________________

Identified Deficiencies:	Proposed Correction:
1. ______________________________	1. ______________________________
2. ______________________________	2. ______________________________
3. ______________________________	3. ______________________________
4. ______________________________	4. ______________________________
5. ______________________________	5. ______________________________
6. ______________________________	6. ______________________________
7. ______________________________	7. ______________________________
8. ______________________________	8. ______________________________
9. ______________________________	9. ______________________________
10. ______________________________	10. ______________________________
11. ______________________________	11. ______________________________
12. ______________________________	12. ______________________________

Programming Nonstructural Cost Estimate

Engineer's Summary Statement:

EVALUATION DATA FORM

RAPID STRUCTURAL EVALUATION EXECUTIVE SUMMARY

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use ____

Seismic Risk Code (W/XXXX/YY YY/ZZZ Z) ________________________________

Evaluator ________________________________ Date ___________________________

Summary of Structural Vulnerability (Wind and Seismic)

Engineering Recommendation for Mitigation

Recommended Continued Evaluation Action

Engineer's Level of Confidence in Rapid Structural Evaluation Results

Recommended Additional Field Work (Brief Scope, Costs and Justification)

Approved by: ___________________________ Date: _____________________

EVALUATION DATA FORM

RAPID NONSTRUCTURAL EVALUATION EXECUTIVE SUMMARY

Base ______________ City ______________ County __________ State ___________

Building ______________ Building Use ______________________________________

Seismic Risk Code (W/XXXX/YY YY/ZZZ Z) ___________________________________

Evaluator ________________________________ Date ___________________________

Summary of Nonstructural Vulnerability (Wind and Seismic)

Engineering Recommendation for Mitigation

Recommended Continued Evaluation Action

Engineer's Level of Confidence in Rapid Nonstructural Evaluation Results

Recommended Additional Field Work (Brief Scope, Costs and Justification)

Approved by: ___________________________ Date: _____________________

WIND EVALUATION STATEMENTS
FOR THE BASIC BUILDING SYSTEM

BUILDING SYSTEM

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1) (Reference 4)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/ suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9) (Reference 3)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d) (Reference 20 & Reference 21)

WIND EVALUATION STATEMENTS
FOR VERTICAL SYSTEMS RESISTING LATERAL WIND FORCES

MOMENT FRAMES

Steel Moment Frames

T F DRIFT CHECK: The building satisfies the Wind Quick Check of the frame drift. (FEMA 178, Sec. 4.2.1)

Concrete Moment Frames

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the average shearing stress in the columns. (FEMA 178, Sec. 4.3.1)

T F DRIFT CHECK: The building satisfies the Wind Quick Check of story drift. (FEMA 178, Sec. 4.3.2)

SHEAR WALLS

Concrete Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the shear walls. (FEMA 178, Sec. 5.1.1)

T F OVERTURNING: All shear walls have hw/lw ratios less than 4 to 1. (FEMA 178, Sec. 5.1.2)

Reinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178, Sec. 5.3.1)

T F REINFORCING: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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 122 millimeters (48 inches); and all vertical bars extend into the reinforced bond beam at the top of the walls. (FEMA 178, Sec. 5.3.2)

Unreinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the unreinforced masonry shear walls. (FEMA 178, Sec. 5.4.1)

T F PROPORTIONS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the height/thickness ratio of the wall panels, between lateral support in either the horizontal or vertical direction, is as follows: (FEMA 178, Sec. 5.5.1)

Allowable Value of Height-to-Thickness Ratio of URM Walls in High Wind Regions

Wall Types	Maximum l/t or h/t	
	Solid or Solid Grouted	All Other
BEARING WALLS		
Walls of one-story buildings	16	13
First-story wall of multistory building	18	15
Walls in top story of multistory building	13	9
All other walls	16	13
NONBEARING WALLS (exterior and Interior[1])	15	13
CANTILEVER WALLS	3	2
PARAPETS	2	1-1/2

[1]Interior wall ratio should be the same as the exterior wall ratio due to the risk of internal pressure through breached openings.

T F PARAPETS: Parapet walls are not less than 203 millimeters (8 inches) thick.

Infill Walls in Frames

T F PROPORTIONS: Unreinforced masonry infill walls satisfy the height-to-thickness criteria for unreinforced masonry nonbearing walls.

T F REINFORCING: Reinforcement in reinforced masonry infill walls satisfies the reinforcing criteria for reinforced masonry walls.

Walls in Metal Buildings

T F LIGHT-GAGE METAL SIDING: Gage of metal siding is adequate for wind loads.

Walls in Wood Frame Buildings

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the wood shear walls. (FEMA 178, Sec. 5.6.1)

T F OPENINGS: Walls with garage doors or other large openings and other walls which would be exposed to internal pressure, as well as external wind forces if the openings are breached, are braced with plywood shear walls or are supported by adjacent construction through substantial positive ties. (FEMA 178, Sec. 5.6.2)

T F WALL REQUIREMENTS: All walls supporting tributary area of 2.23 to 9.3 square meters per 0.35 meters of wall are plywood sheathed with proper nailing or braced and have a height-to-depth (H/D) ratio of 1 to 1 or less or have properly detailed and constructed hold downs. (FEMA 178, Sec. 5.6.3)

T F WALL CONNECTIONS: In high wind regions, wood-frame walls have metal plate connections which comply with the "Deemed-to-Comply Standard" published by the SBCCI (Reference 22). Metal plate connections provide a continuous load path from the foundation to the roof capable of resisting wind uplift.

BRACED FRAMES

Concentric Braced Frames

T F STRESS CHECK: The building satisfies the Wind Quick Check of the stress in the diagonals. (FEMA 178, Sec. 6.1.1)

T F END WALL LATERAL BRACING: End walls are not braced by purlins or girts subject to bending/buckling by wind uplift on the roof or pressure/suction on the sidewalls.

WIND EVALUATION STATEMENTS
FOR DIAPHRAGMS

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consist of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

WIND EVALUATION STATEMENTS
FOR STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F MASONRY WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the bond beams at the top of masonry walls are connected to the vertical wall reinforcing to prevent uplift of the upper portion of the wall.

T F WOOD FRAME WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), steel seismic and hurricane ties provide a continuous link from the roof to the foundation using wood member in tension and fastening the joints with timber connectors at the roof-to-wall, floor-to-floor, and floor-to-foundation locations.

T F METAL PLATE CONNECTED WOOD TRUSSES: There is no evidence to indicate that metal plate connected wood trusses were not designed and constructed in accordance with the recommendations and design criteria of the Truss Plate Institute. (TPI-78) (Reference 23)

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 1.22 meters (4 feet) or less. (FEMA 178, Sec. 8.2.4)

T F TILT-UP WALLS: Precast walls are connected to the diaphragms for out-of-plane wind loads; steel anchors or straps are embedded in the walls and developed into the diaphragm. (FEMA 178, Sec. 8.2.5)

T F GABLE ENDS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), gable ends are adequately attached to the wall and braced against wind loads independent of the roof sheathing or decking.

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation to resist wind uplift and overturning forces. (FEMA 178, Sec. 8.4.1)

T F CONCRETE COLUMNS: All longitudinal column steel is doweled into the foundation. (FEMA 178, Sec. 8.4.2)

T F WOOD POSTS: There is positive connection of wood posts to the foundation and the elements being supported. (FEMA 178, Sec. 8.4.3)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

T F WOOD SILLS: All wall elements are bolted to the foundation sill at 4-foot spacing or less with proper edge distance for concrete and wood. (FEMA 178, Sec. 8.4.7)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS
FOR FOUNDATIONS AND GEOLOGIC SITE HAZARDS

CAPACITY OF FOUNDATIONS FOR WIND UPLIFT

T F OVERTURNING AND UPLIFT: The foundation is capable of resisting the uplift and overturning forces due to wind pressures on the structure.

T F UPLIFT FORCE ON DEEP FOUNDATIONS: Piles and piers are capable of resisting the uplift and overturning forces due to wind pressures on the structure.

T F POLE BUILDINGS: Pole foundations have adequate embedment to resist the uplift and overturning forces due to wind pressures on the structure.

WIND EVALUATION STATEMENTS FOR NONSTRUCTURAL ELEMENTS

The evaluation of nonstructural components should be accomplished with special attention given to the effect of negative and positive pressure. Procedures for doing nonstructural evaluations are described in other publications.

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 1:
WOOD, LIGHT FRAME

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

Walls in Wood Frame Buildings

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the wood shear walls. (FEMA 178, Sec. 5.6.1)

T F OPENINGS: Walls with garage doors or other large openings and other walls which would be exposed to internal pressure, as well as external wind forces if the openings are breached, are braced with plywood shear walls or are supported by adjacent construction through substantial positive ties. (FEMA 178, Sec. 5.6.2)

T F WALL REQUIREMENTS: All walls supporting tributary area of 2.23 to 9.3 square meters per 0.35 meters of wall are plywood sheathed with proper nailing or braced and have a height-to-depth (H/D) ratio of 1 to 1 or less or have properly detailed and constructed hold downs. (FEMA 178, Sec. 5.6.3)

T F WALL CONNECTIONS: In high wind regions, wood frame walls have metal plate connections which comply with the "Deemed-to-Comply Standard" published by the SBCCI (Reference 22). Metal plate connections provide a continuous load path from the foundation to the roof capable of resisting wind uplift.

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F WOOD FRAME WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), steel seismic and hurricane ties provide a continuous link from the roof to the foundation using wood member in tension and fastening the joints with timber connectors at the roof-to-wall, floor-to-floor, and floor-to-foundation locations.

T F METAL PLATE CONNECTED WOOD TRUSSES: There is no evidence to indicate that metal plate connected wood trusses were not designed and constructed in accordance with the recommendations and design criteria of the Truss Plate Institute. (TPI-78)

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls or foundations are spaced at 1.22 meters (4 feet) or less. (FEMA 178, Sec. 8.2.4)

T F GABLE ENDS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), gable ends are adequately attached to the wall and braced against wind loads independent of the roof sheathing or decking.

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WOOD POSTS: There is positive connection of wood posts to the foundation and the elements being supported. (FEMA 178, Sec. 8.4.3)

T F WOOD SILLS: All wall elements are bolted to the foundation sill at 4-foot spacing or less with proper edge distance for concrete and wood. (FEMA 178, Sec. 8.4.7)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 2: WOOD, COMMERCIAL AND INDUSTRIAL

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the wood shear walls. (FEMA 178, Sec. 5.6.1)

T F OPENINGS: Walls with garage doors or other large openings and other walls which would be exposed to internal pressure, as well as external wind forces if the openings are breached, are braced with plywood shear walls or are supported by adjacent construction through substantial positive ties. (FEMA 178, Sec. 5.6.2)

T F WALL REQUIREMENTS: All walls supporting tributary area of 2.23 to 9.3 square meters per 0.35 meters of wall are plywood sheathed with proper nailing or braced and have a height-to-depth (H/D) ratio of 1 to 1 or less or have properly detailed and constructed hold downs. (FEMA 178, Sec. 5.6.3)

T F WALL CONNECTIONS: In high wind regions, wood frame walls have metal plate connections which comply with the "Deemed-to-Comply Standard" published by the SBCCI (Reference 22). Metal plate connections provide a continuous load path from the foundation to the roof capable of resisting wind uplift.

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F WOOD FRAME WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), steel seismic and hurricane ties provide a continuous link from the roof to the foundation using wood member in tension and fastening the joints with timber connectors at the roof-to-wall, floor-to-floor, and floor-to-foundation locations.

T F METAL PLATE CONNECTED WOOD TRUSSES: There is no evidence to indicate that metal plate connected wood trusses were not designed and constructed in accordance with the recommendations and design criteria of the Truss Plate Institute. (TPI-78)

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls or foundations are spaced at 1.22 meters (4 feet) or less. (FEMA 178, Sec. 8.2.4)

T F GABLE ENDS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), gable ends are adequately attached to the wall and braced against wind loads independent of the roof sheathing or decking.

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WOOD POSTS: There is positive connection of wood posts to the foundation and the elements being supported. (FEMA 178, Sec. 8.4.3)

T F WOOD SILLS: All wall elements are bolted to the foundation sill at 4-foot spacing or less with proper edge distance for concrete and wood. (FEMA 178, Sec. 8.4.7)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 3: STEEL MOMENT FRAME

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MOMENT FRAMES

T F DRIFT CHECK: The building satisfies the Wind Quick Check of the frame drift. (FEMA 178, Sec. 4.2.1)

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

VERTICAL COMPONENTS

T F STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation to resist wind uplift and overturning forces. (FEMA 178, Sec. 8.4.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 4:
STEEL BRACED FRAME

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

BRACED FRAMES

T F STRESS CHECK: The building satisfies the Wind Quick Check of the stress in the diagonals. (FEMA 178, Sec. 6.1.1)

T F END WALL LATERAL BRACING: End walls are not braced by purlins or girts subject to bending/buckling by wind uplift on the roof or pressure/suction on the sidewalls.

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

VERTICAL COMPONENTS

T F STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation to resist wind uplift and overturning forces. (FEMA 178, Sec. 8.4.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 5: STEEL LIGHT FRAME

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MOMENT FRAMES

T F DRIFT CHECK: The building satisfies the Wind Quick Check of the frame drift. (FEMA 178, Sec. 4.2.1)

BRACED FRAMES

T F STRESS CHECK: The building satisfies the Wind Quick Check of the stress in the diagonals. (FEMA 178, Sec. 6.1.1)

T F END WALL LATERAL BRACING: End walls are not braced by purlins or girts subject to bending/buckling by wind uplift on the roof or pressure/suction on the sidewalls.

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

VERTICAL COMPONENTS

T F STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation to resist wind uplift and overturning forces. (FEMA 178, Sec. 8.4.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

**WIND EVALUATION STATEMENTS FOR BUILDING TYPE 6:
STEEL FRAME WITH CONCRETE SHEAR WALLS**

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the shear walls. (FEMA 178, Sec. 5.1.1)

T F OVERTURNING: All shear walls have hw/lw ratios less than 4 to 1. (FEMA 178, Sec. 5.1.2)

INFILL WALLS IN FRAMES

T F PROPORTIONS: Unreinforced masonry infill walls satisfy the height-to-thickness criteria for unreinforced masonry nonbearing walls.

T F REINFORCING: Reinforcement in reinforced masonry infill walls satisfies the reinforcing criteria for reinforced masonry walls.

BRACED FRAMES

T F END WALL LATERAL BRACING: End walls are not braced by purlins or girts subject to bending/buckling by wind uplift on the roof or pressure/suction on the sidewalls.

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

VERTICAL COMPONENTS

T F STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation to resist wind uplift and overturning forces. (FEMA 178, Sec. 8.4.1)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

**WIND EVALUATION STATEMENTS FOR BUILDING TYPE 7:
STEEL FRAME WITH INFILL MASONRY SHEAR WALLS**

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

SHEAR WALLS

Reinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178, Sec. 5.3.1)

T F REINFORCING: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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 122 millimeters (48 inches); and all vertical bars extend into the reinforced bond beam at the top of the walls. (FEMA 178, Sec. 5.3.2)

Unreinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the unreinforced masonry shear walls. (FEMA 178, Sec. 5.4.1)

T F PROPORTIONS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the height/thickness ratio of the wall panels, between lateral support in either the horizontal or vertical direction, is as follows: (FEMA 178, Sec. 5.5.1)

Allowable Value of Height-to-Thickness Ratio of URM Walls in High Wind Regions

Wall Types	Maximum l/t or h/t	
	Solid or Solid Grouted	All other
BEARING WALLS		
Walls of one-story buildings	16	13
First story wall of multistory building	18	15
Walls in top story of multistory building	13	9
All other walls	16	13
NONBEARING WALLS (Exterior and Interior[1])	15	13
CANTILEVER WALLS	3	2
PARAPETS	2	1-1/2

[1]Interior wall ratio should be the same as the exterior wall ratio due to the risk of internal pressure through breached openings.

INFILL WALLS IN FRAMES

T F PROPORTIONS: Unreinforced masonry infill walls satisfy the height-to-thickness criteria for unreinforced masonry nonbearing walls.

T F REINFORCING: Reinforcement in reinforced masonry infill walls satisfies the reinforcing criteria for reinforced masonry walls.

BRACED FRAMES

T F END WALL LATERAL BRACING: End walls are not braced by purlins or girts subject to bending/buckling by wind uplift on the roof or pressure/suction on the sidewalls.

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

VERTICAL COMPONENTS

T F STEEL COLUMNS: The columns in the lateral-force-resisting frames are substantially anchored to the building foundation to resist wind uplift and overturning forces. (FEMA 178, Sec. 8.4.1)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 8: CONCRETE MOMENT FRAME

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MOMENT FRAMES

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the average shearing stress in the columns. (FEMA 178, Sec. 4.3.1)

T F DRIFT CHECK: The building satisfies the Wind Quick Check of story drift. (FEMA 178, Sec. 4.3.2)

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls or frame is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

VERTICAL COMPONENTS

T F CONCRETE COLUMNS: All longitudinal column steel is doweled into the foundation. (FEMA 178, Sec. 8.4.2)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 9: CONCRETE SHEAR WALLS

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the shear walls. (FEMA 178, Sec. 5.1.1)

T F OVERTURNING: All shear walls have hw/lw ratios less than 4 to 1. (FEMA 178, Sec. 5.1.2)

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls or frame is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 10: CONCRETE FRAME WITH INFILL SHEAR WALLS

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

Concrete Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the shear walls. (FEMA 178, Sec. 5.1.1)

T F OVERTURNING: All shear walls have hw/lw ratios less than 4 to 1. (FEMA 178, Sec. 5.1.2)

Reinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178, Sec. 5.3.1)

T F REINFORCING: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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 122 millimeters (48 inches); and all vertical bars extend into the reinforced bond beam at the top of the walls. (FEMA 178, Sec. 5.3.2)

Unreinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the unreinforced masonry shear walls. (FEMA 178, Sec. 5.4.1)

T F PROPORTIONS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the height/thickness ratio of the wall panels, between lateral support in either the horizontal or vertical direction, is as follows: (FEMA 178, Sec. 5.5.1)

Allowable Value of Height-to-Thickness Ratio of URM Walls in High Wind Regions

Wall Types	Maximum l/t or h/t	
	Solid or Solid Grouted	All other
BEARING WALLS		
Walls of one-story buildings	16	13
First story wall of multistory building	18	15
Walls in top story of multistory building	13	9
All other walls	16	13
NONBEARING WALLS (Exterior and Interior[1])	15	13
CANTILEVER WALLS	3	2
PARAPETS	2	1-1/2

[1]Interior wall ratio should be the same as the exterior wall ratio due to the risk of internal pressure through breached openings.

INFILL WALLS IN FRAMES

T F PROPORTIONS: Unreinforced masonry infill walls satisfy the height-to-thickness criteria for unreinforced masonry nonbearing walls.

T F REINFORCING: Reinforcement in reinforced masonry infill walls satisfies the reinforcing criteria for reinforced masonry walls.

CONCRETE MOMENT FRAMES

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the average shearing stress in the columns. (FEMA 178, Sec. 4.3.1)

T F DRIFT CHECK: The building satisfies the Wind Quick Check of story drift. (FEMA 178, Sec. 4.3.2)

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls or frame is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F CONCRETE COLUMNS: All longitudinal column steel is doweled into the foundation. (FEMA 178, Sec. 8.4.2)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

**WIND EVALUATION STATEMENTS FOR BUILDING TYPE 11:
PRECAST/TILT-UP CONCRETE WALLS
WITH LIGHTWEIGHT FLEXIBLE DIAPHRAGM**

BUILDING SYSTEM

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the shear walls. (FEMA 178, Sec. 5.1.1)

T F OVERTURNING: All shear walls have hw/lw ratios less than 4 to 1. (FEMA 178, Sec. 5.1.2)

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F TILT-UP WALLS: Precast walls are connected to the diaphragms for out-of-plane wind loads; steel anchors or straps are embedded in the walls and developed into the diaphragm. (FEMA 178, Sec. 8.2.5)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 12: PRECAST CONCRETE FRAMES WITH CONCRETE SHEAR WALLS

BUILDING SYSTEM

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/ lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the shear walls. (FEMA 178, Sec. 5.1.1)

T F OVERTURNING: All shear walls have h_w/l_w ratios less than 4 to 1. (FEMA 178, Sec. 5.1.2)

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F TILT-UP WALLS: Precast walls are connected to the diaphragms for out-of-plane wind loads; steel anchors or straps are embedded in the walls and developed into the diaphragm. (FEMA 178, Sec. 8.2.5)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than 203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

BUILDING SYSTEM

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

Reinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178, Sec. 5.3.1)

T F REINFORCING: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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 122 millimeters (48 inches); and all vertical bars extend into the reinforced bond beam at the top of the walls. (FEMA 178, Sec. 5.3.2)

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F MASONRY WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the bond beams at the top of masonry walls are connected to the vertical wall reinforcing to prevent uplift of the upper portion of the wall.

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 1.22 meters (4 feet) or less. (FEMA 178, Sec. 8.2.4)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

**WIND EVALUATION STATEMENTS FOR BUILDING TYPE 14:
REINFORCED MASONRY BEARING WALLS
WITH PRECAST CONCRETE DIAPHRAGMS**

BUILDING SYSTEM

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

T F HANGAR DOORS: Hangar doors are strong enough and stiff enough to resist wind loads without collapsing or being released from supporting tracks due to excessive deflection.

T F HANGAR DOOR SUPPORTS: Supports for tracks of hangar doors are stiff enough to resist wind uplift on roof without allowing door lateral support to fail, i.e. hangar roof trusses may deflect upward under wind uplift to allow doors to come out of tracks at the top or bottom.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

SHEAR WALLS

Reinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178, Sec. 5.3.1)

T F REINFORCING: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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 122 millimeters (48 inches); and all vertical bars extend into the reinforced bond beam at the top of the walls. (FEMA 178, Sec. 5.3.2)

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F MASONRY WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the bond beams at the top of masonry walls are connected to the vertical wall reinforcing to prevent uplift of the upper portion of the wall.

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 1.22 meters (4 feet) or less. (FEMA 178, Sec. 8.2.4)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

WIND EVALUATION STATEMENTS FOR BUILDING TYPE 15: UNREINFORCED MASONRY BEARING WALL BUILDINGS

BUILDING SYSTEM

T F LOAD PATH: The structure contains a complete load path for wind force effects from any horizontal direction and wind uplift/pressure on the roof perpendicular to the roof surface that serves to transfer the wind forces from the building envelope to the structure and the foundation. (Note: Write a brief description of this linkage for each principal direction including uplift and overturning.) (FEMA 178, Sec. 3.1)

T F EXPOSURE TO INTERNAL PRESSURE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the building envelope encloses the building and has the capacity to remain intact under high wind loads without breaching and subjecting the building to high internal pressure/suction, as well as external pressure/suction. There are no large openings, overhead doors, or glazed openings not designed to resist high wind loads and wind-borne debris. (ASCE 7-95, Sec. 6.7 and Table 9)

CONFIGURATION

T F ROOF OVERHANGS: Roof overhangs (including open porches and carports) do not extend more than 0.61 meters (2 feet) from the exterior envelope of the enclosed building. Soffit enclosures have sufficient strength to protect upward pressures from being applied directly to the underside of the roof decking/sheathing.

LARGE OPENINGS

T F OVERHEAD DOORS: Overhead doors are strong enough and stiff enough to resist wind loads without coming out of tracks and tracks are adequately attached to the door frame/building to resist wind loads.

ADJACENT BUILDINGS

T F ADJACENT BUILDINGS: There is no adjacent or nearby structure, utility/lighting poles, or trees which will potentially collapse under wind loads endangering the building being evaluated or resulting in hazardous missiles, such as roof gravel or ballast, concrete or clay tile roofs, or metal siding or roofing.

MATERIALS AND CONDITIONS

T F OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178, Sec. 3.5.2)

T F POWER DRIVEN NAILS: There is no evidence of pneumatic or mechanically driven staples, nails, P-nails, or allied fasteners. (CABO NER-272 and HUD-FHA UM-25d)

SHEAR WALLS

Unreinforced Masonry Shear Walls

T F SHEARING STRESS CHECK: The building satisfies the Wind Quick Check of the shearing stress in the unreinforced masonry shear walls. (FEMA 178, Sec. 5.4.1)

T F PROPORTIONS: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the height/thickness ratio of the wall panels, between lateral support in either the horizontal or vertical direction, is as follows: (FEMA 178, Sec. 5.5.1)

Allowable Value of Height-to-Thickness Ratio of URM Walls in High Wind Regions

Wall Types	Maximum l/t or h/t	
	Solid or Solid Grouted	All Other
BEARING WALLS		
Walls of one story buildings	16	13
First story wall of multi-story building	18	15
Walls in top story of multistory building	13	9
All other walls	16	13
NONBEARING WALLS (Exterior and Interior[1])	15	13
CANTILEVER WALLS	3	2
PARAPETS	2	1-1/2

[1]Interior wall ratio should be the same as the exterior wall ratio due to the risk of internal pressure through breached openings.

T F PARAPETS: Parapet walls are not less than 203 millimeters (8 inches) thick.

WOOD DIAPHRAGMS

T F SHEATHING: None of the diaphragms consists of straight-laid sheathing or has a span/depth ratio greater than 2 to 1. (FEMA 178, Sec. 7.2.1)

T F SPANS: All diaphragms with spans greater than 7.3 meters (24 feet) have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod braced systems. (FEMA 178, Sec. 7.2.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 12.2 meters (40 feet) and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.2.3)

METAL DECK DIAPHRAGMS

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than 12.2 meters (40 feet) in high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), and have span/depth ratios less than or equal to 3 to 1. (FEMA 178, Sec. 7.3.1)

T F LIGHT-GAGE METAL ROOFING: Gage of metal roofing is adequate for wind loads.

PRECAST CONCRETE DIAPHRAGMS

T F TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), precast concrete diaphragm elements are interconnected by a reinforced concrete topping slab. (FEMA 178, Sec. 7.5.1)

T F CONTINUITY OF TOPPING SLAB: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), 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. (FEMA 178, Sec. 7.5.2)

STRUCTURAL CONNECTIONS

ANCHORAGE FOR WIND UPLIFT FORCES

T F DIAPHRAGM CONNECTION: The connection of the diaphragm to the walls is capable of resisting wind uplift including the effects of internal pressure. (ASCE 7-95)

T F MASONRY WALL ANCHORAGE: In high wind regions (V greater than or equal to 177 km/h (110 mph) or within 161 kilometers (100 miles) of the hurricane oceanline), the bond beams at the top of masonry walls are connected to the vertical wall reinforcing to prevent uplift of the upper portion of the wall.

T F FRAMING CONNECTIONS: Attachment of roof framing to walls is adequate to prevent uplift of roof and provide lateral support to the top of the walls.

ANCHORAGE FOR NORMAL FORCES

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 1.22 meters (4 feet) or less. (FEMA 178, Sec. 8.2.4)

SHEAR TRANSFER

T F TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to shear walls. (FEMA 178, Sec. 8.3.1)

VERTICAL COMPONENTS

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (FEMA 178, Sec. 8.4.4)

T F SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (FEMA 178, Sec. 8.4.5)

T F WALL PANELS: The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178, Sec. 8.4.6)

INTERCONNECTION OF ELEMENTS

T F GIRDERS: Girders supported by walls or pilasters have anchors capable of resisting the wind uplift. Anchors into masonry bond beams provide a positive connection to bond beam reinforcing. (FEMA 178, Sec. 8.5.1)

ROOF DECKING AND WALL PANELS

T F 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 each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.1)

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS WALL PANELS: All light-gage metal, plastic, or cementitious wall panels are properly connected to the wall framing at each supporting member with spacing not greater than <u>203 millimeters (8 inches) on centers at ends of sheets and 305 millimeters (12 inches) on centers at intermediate supports</u>. (FEMA 178, Sec. 8.6.2)

A8.1. Introduction. The detailed structural evaluation is a supplementary step to the rapid structural evaluation when deficiencies discovered cannot be fully described; or the deficiencies cannot be described as life-safety risk and further evaluation of component systems is required. The detailed structural evaluation shall follow the publication of the rapid structural evaluation report and its review by the acceptance authority.

A8.2. Wind Detailed Structural Evaluation. The wind detailed structural evaluation procedure deals principally with life-safety objectives; it does not address other objectives of code compliance or damage control. Hence, the wind evaluation included in this ETL will be limited to the Main Wind Force Resisting System (MWFRS) and those components essential to the stability of the building. This evaluation will not include other components and cladding which, if failure occurs, may expose the building to serious wind and water damage to such non-structural elements as finishes and contents.

A8.2.1. Code Design Provisions Used. The wind detailed structural evaluation will follow evaluation provisions explicit in the standard building code design procedures and standards of ASCE 7-95 (Reference 3). The approximate wind demands are determined in accordance with Attachment 4, *Wind Effects and Force Demands*, of this ETL and ASCE 7-95. The wind forces applied are code required forces as used in building design equation(s) with allowable stresses or load factors which lead to structural capacities (resistance) that are substantially higher than required to resist the code required wind speeds and associated forces. This is different from the approach used for seismic design, where the actual earthquake forces and deflections may be larger than code forces and deflections, but a building will survive by dissipating energy in the yielding of its components if the code provisions concerning force level and detailing have been applied properly.

A8.2.2. Detailed Versus Rapid Structural Evaluation. The provisions used in this attachment for wind detailed structural evaluation are the same as prescribed in Attachment 5, except that the rapid structural evaluation methodology is more approximate.

A8.2.3. Drift. Drift ratios for wind evaluation shall not exceed 0.002, 0.004, and 0.005 for Category I, III, and IV buildings, respectively. These are the same limits prescribed in Attachment 5, *Rapid Seismic and Wind Structural Evaluation*.

A8.2.4. Demand. For wind evaluation using the procedures of this ETL, the demand is based upon the code-required wind speed, pressures, and forces as described in Attachment 4, *Wind Effects and Force Demands*. See Table 3, Performance Requirements for Wind, in this ETL.

A8.2.5. Capacity. The criteria for acceptance under these evaluation procedures will be different from code criteria as follows:

A8.2.5.1. High Risk and Other Buildings (Performance Objective Category III and IV). For these buildings, the code level demand will be evaluated against an ultimate-strength capacity basis obtained by using the procedures of the material chapters of the 1994 *NEHRP Recommended Provisions* (Reference 8) and FEMA 178, Sec. 2.4.9 (Reference 4); i.e., converting to nominal strengths by multiplying working stresses by factors given for the various materials (for example ASD allowable stresses times 1.7 for steel) without using the capacity reduction factor, ϕ.

A8.2.5.2. Immediate Occupancy Buildings (Performance Objective Category I). Buildings in this category will be evaluated using code level demand and code level capacity; i.e., allowable stresses increased by one-third for wind or factored wind loads for strength design.

A8.3. Seismic Detailed Structural Evaluation. The seismic evaluation provisions of this attachment are largely based on AFJMAN 32-1049V2 (Reference 9). They include two methods for the post-yield analysis: Method 1, elastic analysis procedure and Method 2, capacity spectrum method.

A8.3.1. Elastic Analysis Procedure. This is the most commonly used method and is the primary method used in these evaluation provisions. In some references, this method is referred to as the capacity/demand (C/D) method. It uses linear-elastic dynamic analysis to estimate the inelastic performance of a structure. The use of these provisions is appropriately simplified for evaluation by the use of "single mode" assumptions and approximate stress analysis procedures as discussed in Attachment 9 of this ETL. The procedure is based on an element-by-element evaluation rather than on the performance of the building as a complete structure. The method is rather straight-forward and easier to use, but tends to overemphasize individual component behavior, while giving little attention to the interaction between the individual components and their respective forces and moments. As a consequence, it can lead to conservative estimates of vulnerability.

A8.3.2. Capacity Spectrum Method. This method examines the lateral strength of the building as a system, and determines through an incremental collapse analysis, the load-deformation characteristics of the building up to collapse. The structure is periodically modified to include hinges to represent plastic yielding that occurs as the deformation process is continued. The fraction of the evaluation earthquake that can be resisted without collapse is then the indicator of the need for retrofitting and the extent of strengthening needed. Deformation capacity is emphasized rather than strength. Although somewhat more difficult to apply, this method is less conservative and should indicate less need for expensive retrofitting.

A8.3.2.1. The capacity spectrum method is used in conducting detailed structural evaluation when the elastic analysis procedure (capacity/demand method) has been applied and found to be unsatisfactory by criteria of paragraph A8.13.4.6. Both methods will be described in later paragraphs of this attachment.

A8.4. Seismic Building Performance. The deformation ranges in the idealized force-displacement curve of Figure A8.1 show the association of the performance categories of ETL 93-3 (Reference 2). This curve is an idealized graphical representation of the horizontal force (base shear) on the building versus the horizontal displacement (roof displacement) showing capacity designations of the building at several points along the curve (i.e., elastic limit, major yielding, initial deterioration, and ultimate limit of stability). The lines between the points designate the deformation ranges corresponding to each of the performance goals as defined in Table 1 of this ETL. As such, the curve represents the seismic capacity of the building in terms of base shear and roof displacement. Conceptually, the building is deemed to satisfy the performance requirements if the demands of the earthquake do not exceed the capacity of the building at the appropriate range. The procedures to evaluate the adequacy of this capacity in the cases of EQ-I, EQ-II, and EQ-III are prescribed in this attachment. Although the shapes of capacity curves for any building will be unique, they generally will have the following ranges of performance.

A8.4.1. Service Range A. This range covers the capacity of the building up to the lateral force that causes an element to reach its elastic limit (point 1 on Figure A8.1). This point on the capacity curve, which extends beyond the point that represents the design lateral force prescribed in AFJMAN 32-1049V1 (i.e., conventional seismic building code provisions), will be determined by the member strengths of the structural elements as defined in paragraph A8.9.4. Performance within this range of the capacity curve results in no significant structural damage and is consistent with the performance goals for EQ-I.

A8.4.2. Yield Range B. This range covers the capacity of the building from initial yield to major yield (point 2 of Figure A8.1). This point on the capacity curve represents a condition where some of the major lateral-force-resisting elements are beginning to yield and form plastic hinges. As these hinges form, redistribution of the forces will occur and the structure will remain essentially or nearly elastic. Performance within this range of the capacity of the structure is consistent with

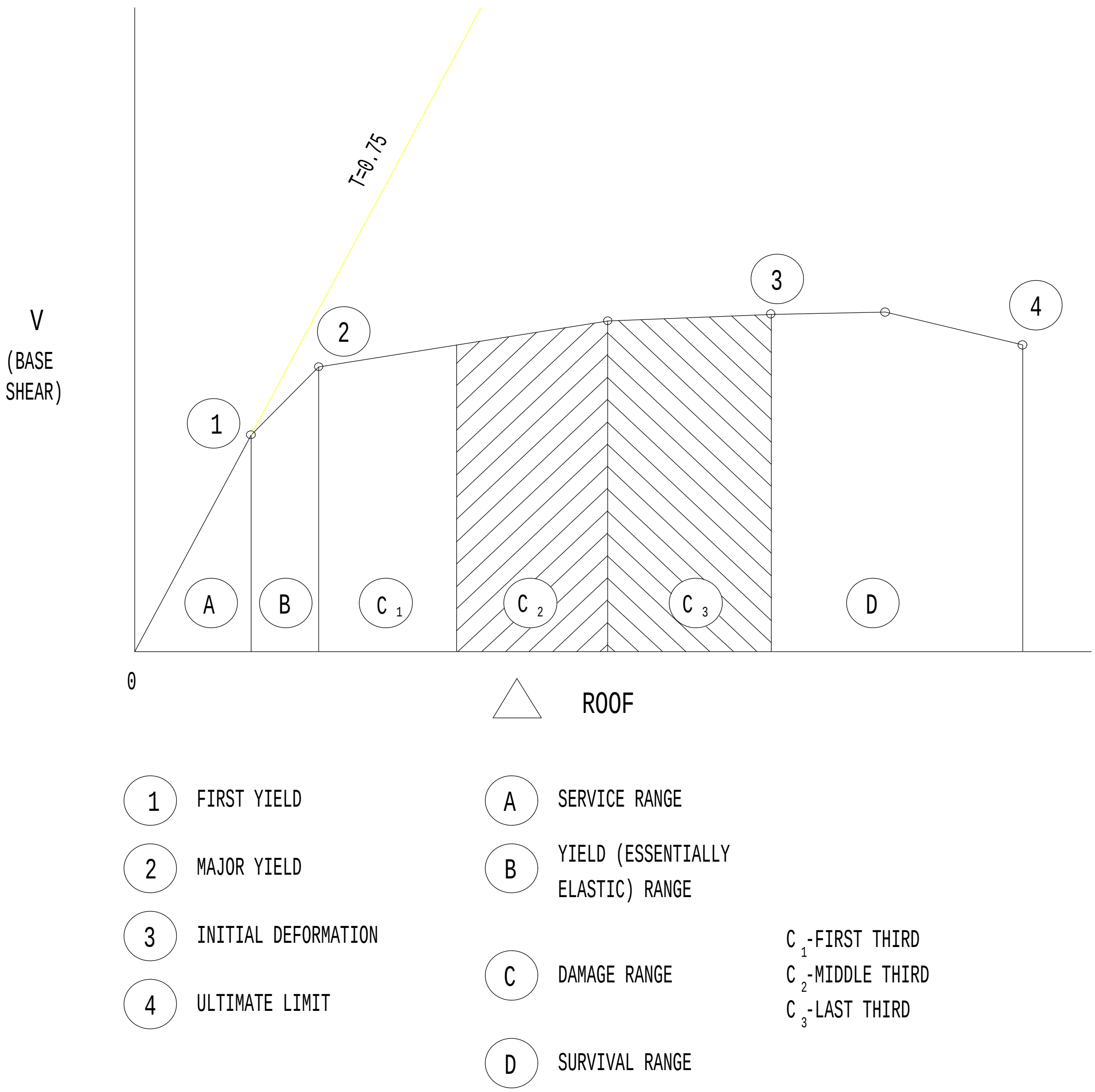


Figure A8.1. Seismic Capacity Curve (AFJMAN 32-1049V2 Draft Revision)

performance standards for nonseismic loads; i.e., no element will exceed its specified strength.

A8.4.3. Damage Range C. This range covers the capacity of the building from major yielding to initial deterioration (point 3 on Figure A8.1). This point on the capacity curve represents a limiting distortion of the structure such that there is no significant loss of capacity due to repeated cyclic loading. In other words, if the structure were laterally displaced to this limit, although there may be evidence of localized structural damage, the building will still be stable and can be occupied. This range represents a fairly wide variation of damage states, depending on ductility, redundancy, and overall behavioral characteristics of the materials and the structural system. Therefore, it is divided into the following three subranges of capacity which are consistent with the performance goals of EQ-II as stated in Table 1 of this ETL.

A8.4.3.1. Subrange C_1 -- Controlled Damage. At this capacity range, it is assumed that there can be continued occupancy and that the damage is easily repairable.

A8.4.3.2. Subrange C_2 -- Maintain Function. At this capacity range, it is assumed that emergency services can be maintained and buildings with large occupancies can be safely exited.

A8.4.3.3. Subrange C_3 -- Life-Safety. At this capacity, it is assumed that life-safety is maintained within the building and that there is adequate containment of hazardous materials. The degree of damage is most likely repairable.

A8.4.4. Survival Range D. This range covers the capacity of the building from initial deterioration to the ultimate limit of stability (point 4 on Figure A8.1). This point on the capacity curve represents a distortion limit prior to a potential of vertical instability and partial collapse of portions of the building. This range of damage is deemed unacceptable for EQ-II, but cautiously acceptable for the possibility of the highly unlikely, low probability of occurrence, EQ-III.

A8.5. Design and Need for Validation. Standard seismic design building code provisions, like those of AFJMAN 32-1049V1, provide detailing requirements which allow buildings to be designed to low force levels and yet sustain yielding in larger earthquakes. The provisions are based on allowable stresses and static lateral force procedures. The prescribed forces and detailing requirements are assumed to produce a structure that will satisfy the intended performance goals for buildings subjected to large earthquakes. No knowledge is provided of the actual performance that can be expected of a particular building in an earthquake. Provisions are needed that will verify the performance that is only assumed by the provisions of the standard seismic design building code provisions, like those of AFJMAN 32-1049V1.

A8.5.1. Design and Evaluation. ASCE SC 1-96 (Reference 5) treats this problem using two alternative recommended design procedures. These include the Basic Design Method, a single-level approach, and the Alternate Design Method, a two-level design approach. Both employ the design provisions of the standard model building codes. Similar to the Level B rapid structural evaluation provisions, the Basic Design Method consists of the application of the standard building code provisions, but with the seismic design force calculated based upon two-thirds the value of the spectral accelerations of the maximum earthquake considered (2%/50 years). The Alternate Design Method is similar to the two-level detailed structural evaluation methodology of these provisions. The evaluation and verification of capacity is added as a design requirement to standard seismic design building code provisions. An iterative approach to design is used. First, the building is devised conforming to the standard building code requirements. These incorporate EQ-II, 500-year return period, Table 2 in the ETL. Next, the building is evaluated using added post-yield evaluation procedures for adequacy to meet the required performance standards. The post-yield evaluation provisions are provided in AFJMAN 32-1049V2 (Reference 9). Third, if the building does not meet the required performance standards, the structural design is revised and the verification process repeated. Finally, this process is repeated until the design has converged.

A8.5.2. ETL Evaluation Provisions. The evaluation provisions of this ETL are drawn from AFJMAN 32-1049V2 (Reference 9). Accordingly, they are generic to those provisions that will eventually be used in the design of necessary structural strengthening.

A8.6. Required Types of Evaluation Analysis. Detailed structural evaluation involves conducting analyses for EQ-I, EQ-II, and EQ-III as shown in Figure 3, Seismic Detailed Structural Evaluation Procedures, of this ETL.

A8.6.1. EQ-I Analysis. Analysis for EQ-I is required only for essential facilities. The structure is expected to perform within the yield range in the 70-year earthquake. Obtaining forces this way has two virtues: the spectrum is a better representation of the earthquake than base shear coefficients of the code approach, and the lateral forces so obtained reflect more accurately the dynamic characteristics of the building. The EQ-I procedures are quite ordinary. The analysis is a conventional linear elastic analysis, and the member strengths (e.g., ACI and AISC design strengths) are calculated in the usual way. Some individual members of the building are allowed to have demands exceed capacity within limits as long as they do not affect the overall performance of the building.

A8.6.2. EQ-II Analysis. Analysis for EQ-II is required for all buildings subjected to the requirements of Detailed Structural Evaluation. The performance requirements for acceptable damage and earthquake return period depend on the occupancy category of the building as shown in Table 2, Performance Requirements for Seismic Loads, of this ETL. The earthquake level is comparable to the code earthquake, but it is evaluated by an approximate inelastic procedure rather than a codified elastic procedure. The verification process, the elastic analysis procedure, uses a linear procedure similar to the EQ-I method with modifications. Basically, a structural analysis is performed. Generally, the member forces (i.e., demand) exceed the strengths (i.e., capacity) of the members. The ratio of demand to capacity is calculated for each member. Referred to as the Demand Capacity Ratio (DCR), these values are compared to prescribed IDRs that have been developed for various kinds of members. The IDRs are based on experimental data and/or judgment; they are intended to be conservative because this is a relatively quick procedure. A procedure is given for reviewing the DCRs. If the DCRs are less than the IDRs, and if the building meets the criteria of the review, the verification process ends. If the DCRs exceed the IDRs, the building does not satisfy the evaluation procedure and mitigation is defined and programmed.

A8.6.3. EQ-III Analysis. An evaluation for EQ-III is required when this highly unlikely event is greater than the EQ-II event by a factor of 1.5. If the structure had satisfied the EQ-II criteria by the elastic analysis procedure, a check of DCR values will be made for EQ-III. Justifiable adjustments to relax the IDR requirements may be made as approved by the Project Engineer. Refer to paragraph A8.13.5 for further guidance.

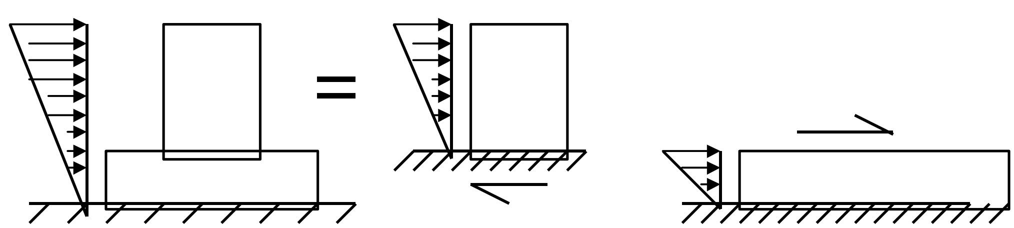
A8.7. Use of Simplified Dynamic Analysis. The static lateral force procedure of building codes attempts to approximate the dynamic analysis procedure. The force distribution in the static lateral force approach approximates the fundamental mode shape of a uniform (regular) building with an F_t adjustment for higher modes. The static lateral force procedure generally gives a fair approximation of a regular, uniform linear dynamic analysis, but is not valid for buildings with irregular features or for buildings that respond in a nonlinear, inelastic manner. Hence, a detailed structural evaluation will be done using dynamic modal analysis, but in a simplified form as discussed in the following paragraphs.

A8.7.1. Attachment 11 describes the procedures of modal analysis. In A11.9, it advises that unless the building is unusual or irregular in elevation or plan, the modal analysis of low-rise buildings up to about five stories can generally be limited to the fundamental mode of vibration. Even when the building is irregular, in certain key situations the approximation can still be applied with reasonable success. Although the use of a computer program will generally be more efficient and give more accurate results, the single mode analysis can be done by hand calculations with sufficient accuracy.

A8.7.2. Practically all buildings to be evaluated under the provisions of this ETL are low-rise buildings less than five stories in height. In keeping with the approximate nature of evaluation prescribed in this ETL, a simplified single mode version of dynamic analysis is adopted for evaluation. The modal equations of Attachment 11 (Equations A11-1 through A11-6) are reduced to the simplified forms provided in section A8.10 for building evaluation. Paragraph A11.9 of Attachment 11 provides useful background description of modal analysis that can be used in evaluating higher buildings (e.g., five to fifteen stories) or those of complex irregularities or mass distributions.

A8.8. Analysis with Irregular Buildings. Air base facilities include irregular buildings. With certain approximations, the procedures of this ETL for regular buildings may be used for the evaluation of those buildings with irregular features. For vertical irregularities, the vertical distribution of the base shear can be revised to match the deflected shape determined from the initial application of the standard distribution. This method is routinely used in the provisions of this ETL to define the first or fundamental mode for base shear and story shear calculations (see A8.10.3).

A8.8.1. Buildings With Vertical Irregularities. For buildings with setbacks there are approximate methods available. For example, for a tower structure on a wide base structure (Figure A8.2), a method can be used that evaluates the two portions in a two-step procedure if the following conditions are (Reference 24): (1) the base portion and the tower portion, considered as separate buildings, can be classified as regular; and (2) the stiffness of the top story of the base is at least five times that of the first story of the tower. When these conditions are met, the base and tower may be analyzed as separate buildings in accordance with the following: (1) the tower may be analyzed in accordance with the single-mode provisions of this ETL with the base taken at the top of the base portion; and (2) the base portion shall then be analyzed in accordance with the same provisions using the height of the base portion and with the gravity load and base shear of the tower portion acting at the top level of the base portion. Figure A8.3 illustrates this modeling for analysis.



ELEVATIONS

Figure A8.2. Tower on a Wide Base Structure

A8.8.2. Buildings With Horizontal Irregularities. For buildings with horizontal irregularities, the initial evaluation can be done by an enveloping procedure where the building is designed twice, once assuming the horizontal irregularities are not significant (i.e., rigid diaphragm, relative rigidities); and then assuming the irregular portions work independently (i.e., discontinuities in the diaphragm and tributary areas in portions of the building) (Figure A8.3). Each structural member is evaluated for the worst case. If the diaphragm is flexible, the irregular portions should be assumed to act only independently. Structurally separated entities of a building must be fully capable of resisting vertical and lateral forces on their own. In addition, the possibility of pounding must be considered in each case.

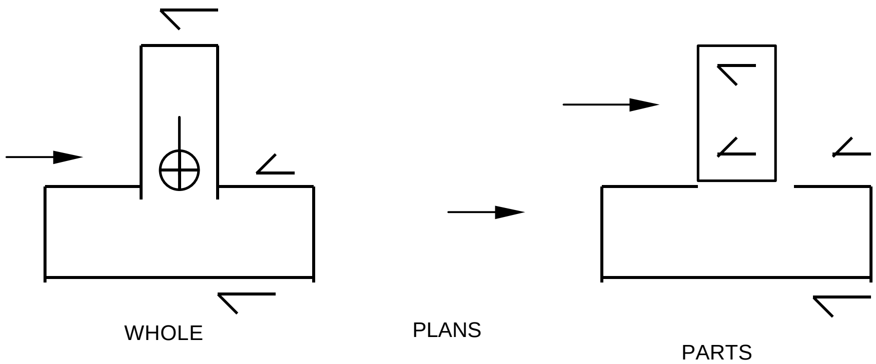


Figure A8.3. Irregularity in Plan: T-Shaped Building

A8.9. Evaluation Criteria.

A8.9.1. Lateral Displacements and Drift Limits.

A8.9.1.1. Drifts. Interstory drifts shall not exceed the values given in Table A8.1 unless it can be demonstrated that greater drifts can be tolerated within the performance goals.

Table A8.1. Drift Ratio Limits

Category	EQ-I	EQ-II	EQ-III
I. Immediate	0.007	0.015	0.020
III. High Risk	N/A	0.020	0.030
IV. Other Building	N/A	0.020	0.030

A8.9.1.2. Building Separations. Building separation is of no concern unless there is an immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being examined. A neighboring structure is considered to be “immediately adjacent” if it is within two inches times the number of stories away from the building being evaluated. Under the conditions of these requirements, some contact between buildings is acceptable if it can be shown that the effects of pounding will not cause loss of function, instability of the affected portion of the structure, or risk to life-safety. For example, if all the floors of adjacent buildings are in vertical alignment with each other, then the pounding associated with the post-yield conditions might cause only some minor local damage to the material in contact. However, if the floor of one building is in alignment with mid-height of columns in the adjacent building, pounding could cause column instability due to buckling and P-Δ effects. If contact is to be avoided, the minimum separation between buildings will be governed by the combined maximum displacements of the adjacent buildings. If multiple modes are considered, the maximum story displacements, at respective locations, may be combined by the square-root-of-the-sum-of-the-squares to determine the minimum separation.

A8.9.1.3. P-Δ Effects. The secondary effects of the lateral displacements (Δ) combined with the gravity forces (P) will be investigated unless the drift satisfies the "Quick Check for Drift" given in Attachment 5. The P-Δ effects in a given story are due to the eccentricity of the gravity loads above the story. If the story drift due to the lateral forces is Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the P-Δ moment to the lateral-force story moment can be designated as a stability coefficient. If the stability coefficient is less than 0.10 for every story, then the P-Δ effects can be considered insignificant. If, however, the stability coefficient exceeds 0.10 for any story, then the P-Δ effects are significant and the building must be programmed for mitigation.

A8.9.2. Overturning. The structure shall be designed to resist the overturning effects of the seismic loading. In some portions of the structure, the resulting forces may cause uplift at the foundation interface, thus creating an apparent condition of instability. However, structures designed for force levels substantially less than those experienced during actual earthquakes have not exhibited this behavior. Although the state of the art of earthquake engineering has not been able to establish a consistent recommendation for evaluating this condition, it is generally acceptable that buildings can be subjected to rocking on their bases, that the resulting displacements do not approach an incipient overturning condition, and that the maximum displacement is limited by the short time interval between load reversals. When the evaluation engineer determines that uplift conditions exist, two basic retrofit choices exist: (1) tie down the foundation to prevent uplift; or (2) do not provide any additional restraint on the potential uplift. The decision requires some judgment of the evaluating engineer. If the foundation is tied down, the resulting forces on the structure will generally be increased in the event of a large earthquake because of the added rigidity of the overall structural system. If uplift is allowed to occur, the resulting seismic forces may actually be reduced because of increased energy absorption and the nonlinearity of the base rocking; however, the redistribution of loads to other portions of the foundation may cause some distress in the structure or at the foundation. When uplift is allowed to occur, the designer should provide justification for the assumed redistribution of loads and for the adequacy of the structure and foundation.

A8.9.3. Horizontal Torsional Moments. Elements that are intended to resist torsion should be located at or near the periphery of the building to maximize torsional rigidity. When this has not been accomplished or when there are large horizontal eccentricities, the structure must be analyzed for potential torsional instability.

A8.9.3.1. Compare the forces due to translational motion to the forces due to torsional motion for all lateral-force-resisting components. If the torsional portion is a substantial amount of the total design force (e.g., one-third of the total), then torsional stability will be evaluated.

A8.9.3.2. Review the mathematical modeling assumptions and calculations to evaluate the validity of the modeling techniques. Determine if uncertainties in assumptions would increase or decrease the torsional characteristics.

A8.9.3.3. Investigate the consequences of the worst case conditions. The torsional moment at a given story is the moment resulting from the eccentricities between the center of the applied lateral forces at levels above that story and the center of rotation of the vertical resisting elements in that story plus an accidental torsional moment. The accidental torsional moment should be determined assuming displacements of the centers of mass each way from their calculated locations. The minimum assumed displacements of the center of mass at each level can be estimated to equal five percent of the dimension at that level measured perpendicular to the direction of the applied force. For each element, the more severe loading should be considered. If a deficiency is defined, the feasibility of revising the lateral-force-resisting system to minimize the effects of horizontal torsional moments will be evaluated in a subsequent phase of evaluation, retrofit concepts and program cost estimates.

A8.9.4. Member Strengths (MS). Calculate element capacities on the ultimate-strength basis of the 1991 *NEHRP Recommended Provisions* (Reference 8). The provisions refer to the national standards and reference documents that should be used. The modifications that are needed to put all materials on an ultimate-strength basis are summarized below. When calculating capacities of deteriorated elements, the evaluating engineer should make appropriate reductions in the material strength, the section properties, and any other aspects of the capacity affected by the deterioration.

A8.9.4.1. Wood. The basic document is the *National Design Specification for Wood Construction* (Reference 25), which is written on a working-stress basis. The 1991 *NEHRP Recommended Provisions* (Reference 8) makes a conversion to ultimate-strength basis by providing capacity reduction factors and by using stresses 2.0 times the allowables.

A8.9.4.2. Steel. The basic documents are *Load and Resistance Factor Design (LFRD), Manual of Steel Construction* (*Part 6 - Structural Steel Buildings*) (Reference 26) including Supplement 1, effective January 1, 1989, and *Allowable Stress Design, Manual of Steel Construction* (*Part 5 - Plastic Design Specification for Structural Steel Buildings*) (ASD) (Reference 27). The use of the latter requires conversion to ultimate strength through multiplication of the allowable stresses by 1.7. Note that the resistance factor, f, is not used in this ETL.

A8.9.4.3. Concrete. The basic document is ACI 318-89 (Reference 28). Because this document is written on an ultimate-strength basis, the 1988 *NEHRP Recommended Provisions* specifies special load factors that include the factor of 1.0 for earthquake effects (see Equations A8-10 and A8-11).

A8.9.4.4. Masonry. The basic document is the ACI-ASCE *Building Code Requirements for Masonry Structures* (Reference 29), with modifications. The 1991 *NEHRP Recommended Provisions* (Reference 8) specifies member strength reduction factors, f, and the use of stresses 2.5 times the allowable stress (f not used in this ETL).

A8.10. Lateral Forces, Displacements, and Drifts.

A8.10.1. Base Shear. The seismic base shear specified below is the basic seismic demand on the building. Element forces and defections obtained from analysis based on this demand are the element demands, (E), to be used in the load combinations of Equations A8-10 and A8-11.

A8.10.1.1. The seismic base shear, (V), in a given direction, should be determined as follows:

$$V = C_S W \qquad \text{(A8-1)}$$

where:

C_S = the seismic design coefficient determined below

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 0.42 kg/m^2 (10 lb/ft^2) of floor area, whichever is greater
- Total operating weight of all permanent equipment
- The effective snow load is equal to either 70 percent of the full design snow load or, where conditions warrant and approved by the Contracting Officer, not less than 20 percent of the full design snow load except that, where the design snow load is less than 1.26 kg/m^2 (30 lb/ft^2), no part of the load need be included in seismic loading.

A8.10.1.2. There are two equations for the seismic coefficient, (C_S). Equation A8-2 depends on the building period, and Equation A8-3, which gives an upper limit to the value of C_S, is applicable to buildings with short periods. The seismic coefficient, (C_S), for existing buildings should be determined as follows:

$$C_S = 0.85\frac{S_{aml}}{T^n}0 \qquad \text{(A8-2)}$$

where:

S_{aml}	=	$F_v S_{ML}$
	=	Design validation spectral acceleration in the long-period range for the maximum earthquake ground motion considered
F_v	=	Site coefficient in the long-period range given in Table A8.3 for the soil profile type defined in Table A8.2
S_{ML}	=	Spectral acceleration in the long-period range for soil profile Type B for the maximum earthquake ground motion considered representing EQ-I, EQ-II, or EQ-III spectral acceleration S_{ML} as provided in Attachment 2
T	=	the fundamental period of the building
n	=	1.0 for T<1.0 second and 2/3 for T>1.0 second

The value of C_S need not be greater than the following

$$C_S = 0.85 S_{ams} 0 \qquad \text{(A8-3)}$$

where:

S_{ams}	=	$F_a S_{MS}$
	=	Design validation spectral acceleration in the short-period range for the maximum earthquake ground motion considered representing EQ-I, EQ-II, or EQ-III spectral acceleration S_{MS} from Attachment 2

F_a = Site coefficient in the short-period range given in Table A8.4 for the soil profile type defined using Table A8.2

S_{MS} = Spectral acceleration in the short-period range for soil profile Type B for the maximum earthquake ground motion considered representing EQ-I, EQ-II, or EQ-III spectral acceleration S_{MS} provided in Attachment 2

A8.10.1.2.1. As an exception to Table A8.2, when the soil properties are not known in sufficient detail to determine the soil profile type, Type D shall be used. Soil profile Types E or F need not be assumed unless the building owner (MAJCOM or base) determines that Types E or F may be present at the site or in the event that Types E or F are established by the geotechnical data.

A8.10.1.2.2. If the $\overline{S}_u$ is used and the $\overline{N}_{ch}$ and $\overline{S}_u$ criteria differ, select the category with the softer soils (for example, use soil profile Type E instead of D).

A8.10.2. Period. For use in Equation A8-2, the value of T should be calculated using one of the following methods.

A8.10.2.1. 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.

A8.10.2.1.1. 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:

$$00\,T_a = C_T h_n^{3/4} \qquad \text{(A8-4a)}$$

where:

C_T = 0.035 for steel frames

C_T = 0.030 for concrete frames

h_n = the height in meters (feet) above the base to the highest level of the building

A8.10.2.1.2. As an alternate for concrete and steel 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 A8-4a.

A8.10.2.1.3. For all other buildings,

$$T_a = \frac{0.05\,h_n}{\sqrt{L}}\,0 \qquad \text{(A8-4b)}$$

Table A8.2. Soil Profile Type Classification

Soil Profile Type	Soil Profile	$\overline{v_s}$	N or N_{ch}	$\overline{s}_u$
A	Hard rock	>1500 m/s (>5,000 ft/sec)		
B	Rock	760 to 1500 m/s (2,500 to 5,000 ft/sec)		
C	Very dense soil and soft rock	360 to 760 m/s (1,200 to 2,500 ft/sec)	>50	≥100 kPa (2,000 lb/ft^2)
D	Stiff soil	180 to 360 m/s (600 to 1,200 ft/sec)	15 to 50	50 to 100 kPa (1,000 to 2,000 lb/ft^2)
E	Soil	<180 m/s (<600 ft/sec) or any profile with more than 3 m (10 ft) of soft clay defined as soil with PI>20, w>40%, and s_u <25 kPa (500 lb/ft^2)	<15	<50 kPa (<1,000 lb/ft^2)
F	A soil profile requiring site-specific evaluations: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils 2. Peats and/or highly organic clays (H>3 m [10 ft] of peat and/or highly organic clay where H=thickness of soil) 3. Very high plasticity clays (H>25 ft [8 m] with PI>75) 4. Very thick soft/medium stiff clays (H>36 m [120 ft])			

Note: These soil types are defined in the 1994 *NEHRP Recommended Provisions*.

Table A8.3. Values of F_v For Class B Sites

Site Class	Design Spectral Acceleration at 1 Second for Class B Sites				
	$S_{ML} \le 0.1$	$S_{ML}=0.2$	$S_{ML}=0.3$	$S_{ML}=0.4$	$S_{ML} \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a
F	a	a	a	a	a

Note: Use straight line interpolation for intermediate values of S_{ML}. Site specific geotechnical investigation and dynamic site response analysis shall be performed.

Table A8.4. Values of F_a For Class B Sites

Site Class	Design Spectral Acceleration at Short Periods for Class B Sites				
	$S_{MS} \le 0.25$	$S_{MS}=0.50$	$S_{MS}=0.75$	$S_{MS}=1.00$	$S_{MS} \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a
F	a	a	a	a	a

Note: Use straight line interpolation for intermediate values of S_{MS}. Site specific geotechnical investigation and dynamic site response analysis shall be performed.

where:

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

A.10.2.2. 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:

$$0\, T = 2p\sqrt{\frac{\Sigma\left(w_i d_i^2\right)}{g\Sigma\left(f_i d_i\right)}} \quad \text{(A8-5)}$$

A8.10.2.2.1. The values of f_i represent any lateral force, associated with weights w_i, distributed approximately in accordance with A8.10.3 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 A8.5.

Table A8.5. Coefficient for Upper Limit on Calculated Period

S_{adl}*	C_a
60	1.2
45	1.3
30	1.4
20	1.5
15	1.7
7.5	1.7

*Expressed in percent of gravity

A8.10.3. Fundamental Mode Shape. The following procedures are to be used to calculate the fundamental mode shape when using single-mode analysis. The fundamental mode shape will be used below to distribute the base shear to the various story levels (A8.10.5). Paragraph A11.9 of Attachment 11 provides guidance for other cases involving multimode analysis. Paragraph A9.8 (Attachment 9) provides an illustrative problem of fundamental mode shape calculation using approximate analysis.

A8.10.3.1. Single Story Building. The modal analysis procedure becomes essentially equivalent to a static design procedure. The seismic design coefficient, C_s, will be equal to the spectral acceleration, S_a. Thus, the total lateral force on the building, for each direction of motion will be equal to the spectral acceleration times the weight of the building ($V=S_aW$) in accordance with Equation A11-4.

A8.10.3.2. Low-Rise Buildings up to Five Stories. The following iterative process is used to determine the mode shape.

A8.10.3.2.1. Assume a mode shape and calculate the quantity f at each level. Start with a straight line, and with f=1 at the roof. The exact shape and amplitude do not matter -- the shape will be refined and the amplitude will be normalized.

A8.10.3.2.2. Apply lateral forces, f_x proportional to w_xf_x, and calculate the deflection.

A8.10.3.2.3. Convert the deflected shape to a mode shape, with f=1 at the roof.

A8.10.3.2.4. Repeat this process until the deflected shape is the same as the assumed mode shape.

A8.10.4. Damping. All of the design spectra given by Equations A8-2 and A8-3 are for structural damping equal to 5 percent of critical damping. These spectra may be converted to other damping ratios by the factors given in Table A8.6. Linear interpolation may be used to provide factors for intermediate damping values. Table A8.7 provides damping values for various structural systems in the elastic-linear and post-yield deformation ranges.

Table A8.6. Damping Adjustment Factors, β

Percent	Multiplying Factor for the 5% Spectrum
2	1.25
5	1.00
7	0.90
10	0.80
15	0.70
20	0.60

Table A8.7. Damping Values for Structural Systems

Structural System	EQ-I Analysis	EQ-II and EQ-III Analysis
Structural steel	3%	7%
Reinforced concrete	5%	10%
Masonry shear walls	7%	12%
Wood	10%	15%
Dual systems	Note 1	Note 2

Notes:

1. Use the value of the primary, or more rigid system. If both systems are participating significantly, a weighted value, proportionate to the relative participation of each system, may be used.
2. The value for the system with the higher damping value may be used.

A8.10.5. Story Forces. The lateral force, F_x, induced at any level should be determined as follows:

$$F_x = C_{vx} V \quad 0 \qquad \text{(A8-6)}$$

$$0\, C_{vx} = \frac{w_x f_x}{\sum_{i=1}^{n} w_i f_i} \quad 0 \qquad \text{(A8-7)}$$

where:

C_{vx} = vertical distribution factor

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

ϕ_i = displacement amplitude of fundamental mode at Level i

ϕ_x = displacement amplitude of fundamental mode at Level x

Note: In the static approach, the base shear is distributed over the height of the building in proportion to wh/Σwh; in the simplified modal analysis approach as shown above, the distribution is made in proportion to wϕ/Σwϕ.

A8.10.6. Horizontal Distribution of Shear. The story shear, V_x, should be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm.

A8.10.7. Displacements and Drift. The lateral displacement at Level x may be taken equal to the displacement of the fundamental mode as determined by the following equation:

$$d_x = \left\{ \frac{\sum_{i=1}^{n} w_i f_i}{\sum_{i=1}^{n} w_i f_i^2} \right\} f_x S_a \left(\frac{T}{2p} \right)^2 g\,0 \qquad \text{(A8-8)}$$

where:

S_a = spectral acceleration

$$S_a = \left(\frac{V}{W} \right) \div a$$

where:

α = effective modal weight for fundamental mode

$$= \frac{\left(\sum_{i=1}^{n} w_i f_i \right)^2}{\sum_{i=1}^{n} w_i \left(\sum w_i f_i^2 \right)}$$

Other parameters are described in A8.10.1, A8.10.2 and A8.10.5.

A8.10.8. Demand on Parts and Portions of the Building. This section addresses those building elements that are not part of the lateral-force-resisting system. These elements include nonstructural architectural and mechanical elements (e.g., appendages, exterior cladding, and equipment) and structural elements that are not part of the lateral-force-resisting system or are part of the lateral-force-resisting system only in the other direction (e.g., walls when considered with an orientation perpendicular to the direction of the earthquake). 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, should be evaluated to verify that they are capable of resisting the seismic forces specified below under the provisions of ETL 97-11, *Mitigation of Non-Structural Seismic and High Wind Deficiencies for Existing Buildings.*

A8.10.8.1. Each element or component evaluated should be capable of resisting a total lateral seismic force, F_p, which is 85 percent of the requirement in FEMA 222, 1991 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings,* as provided below. The formula recommended considers that the probable nonlinear behavior of the lateral-force-resisting system shifts the floor spectra into the velocity range of the spectra. Therefore, use of the reduction for this portion of the spectra is included in the formula.

$$F_p = 0.85\ (0.70\ S_{DL}C_cW_c) = 0.60\ S_{DL}C_cW_c \qquad \text{(A8-9)}$$

where:

S_{DL} = Spectral Acceleration (1.0), S_{DL}, for 500-year earthquake given in Attachment 2

C_c = a coefficient given in Table A8.8

W_c = the weight of the element or component

Table A8.8. Seismic Coefficient, C_c

		C_c
Parts of Structure	Walls: Unbraced (cantilevered parapets and walls) Other exterior walls at and above the ground floor All interior bearing and nonbearing walls and partitions	 2.4 0.9 0.9
	Masonry or concrete fences over 1.8 meters (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 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	 2.4 0.9
	Mechanical, plumbing, and electrical equipment	0.9
	Anchorage for permanent floor-supported cabinets and bookstacks more than 1.5 meters (5 feet) in height (includes contents)	0.9
	Anchorage for suspended ceilings and light fixtures	0.9

A8.11. Modeling. The building will be modeled as a system of masses lumped at each floor level, each mass having one degree of freedom, that of lateral displacement in the direction under consideration. The computer masses will be in conformance with the weights prescribed in AFJMAN 32-1049V1. The stiffness of the lateral-force-resisting system will be determined using methods in accordance with paragraph A8.10.2 and A11.9 of Attachment 11.

A8.12. Analysis for EQ-I. This section prescribes detailed requirements and provides procedures for verification of performance in EQ-I. The structure is required to remain essentially elastic for the required earthquake, using the required forces.

A8.12.1. Nearly Elastic Behavior. Nearly elastic behavior means that the calculated forces on some structural elements slightly exceed the member strength (MS), but that the elastic-linear behavior of the overall structure is not substantially altered (i.e., it is essentially elastic). For a structure that has a multiplicity of structural elements that form the lateral-force-resisting system, the yielding of a small number of elements will generally not affect the overall elastic behavior of the structure if the excess load can be redistributed to other structural elements that are not loaded to their MS. This condition will be considered satisfied if the guidelines given below are met. Table A3.3 should be used to establish the indicated R values.

A8.12.1.1. Ductile Framing Systems. Ductile framing systems are defined as those systems conforming to AFJMAN 32-1049V1 (Reference 7) classifications for R=10 or 12. For these systems, a limited percent of the structural elements that resist lateral forces in flexure in the direction of the force may have load combinations that exceed the flexure MS by a value of up to 25 percent. The percent of horizontal elements having load combinations greater than their flexural MS at any story is limited to 20 percent and the number of vertical elements having load combinations greater than their flexural MS is limited to 10 percent on any story.

A8.12.1.2. Other Framing Systems. Framing systems conforming to AFJMAN 32-1049V1 classifications for R=8 may have a limited percent of the lateral-force-resisting structural elements in each story, in the direction of the force, that have load combinations that exceed the MS by a value up to 10 percent. The number of horizontal elements having load combinations greater than their MS at any story is limited to 20 percent and the number of vertical elements having load combinations greater than their MS at any story is limited to 10 percent.

A8.12.1.3. Box Systems. Lateral-force-resisting systems that have the AFJMAN 32-1049V1 classifications with R less than 8 may not have load combinations that exceed the MS.

A8.12.1.4. Energy Absorbing Elements. In some lateral-force-resisting systems, elements are specifically designed to yield prematurely to act as sacrificial elements. Examples include the link beams in eccentric based frames, beams connecting coupled shear walls, and elements in energy absorbing devices. In the event of a major earthquake, these elements are expected to yield prior to having the lateral forces reach a level that will damage other structural elements. These special energy absorbing elements are not subject to the requirements of this attachment as long as they are in conformance with the requirements of AFJMAN 32-1049V1 (Reference 7).

A8.12.2. Forces. Buildings will be analyzed for the lateral forces derived from the modal analyses using EQ-I of the required return period specified in Table A8.1.

A8.12.3. EQ-I Analysis Modeling. The results of a lateral-force analysis can be very sensitive to the assumptions made for the stiffness of the structural elements when constructing a mathematical model of the structure. As the stiffness is overestimated, the period of vibration shortens and the displacements reduce. However, a shorter period may possibly attract higher forces. When the stiffness is underestimated, periods lengthen, lateral displacements increase, and lateral forces may be reduced. When the relative rigidities of various lateral-force-resisting elements are not accurately utilized, there can be a great amount of uncertainty in the torsional characteristics of the structure. The effects of nonstructural elements, as well as structural elements not part of the lateral-force-resistant system, can have a significant effect on the response of the overall structure to earthquake ground motion. Therefore, it is important to account for possible inaccuracies in the mathematical model. When there are uncertainties, an attempt should be made to envelope the possibilities to assure good performance of the structure in case of an earthquake. The stiffness characteristics may vary with amplitude of lateral motion, thus the model used for an AFJMAN 32-1049V1 analysis may vary from the model that represents the yield level capacity or the ultimate post-yield capacity. For an elastic analysis, the following factors should be considered.

A8.12.3.1. Gross concrete section properties are considered appropriate for modeling the stiffness of reinforced concrete members.

A8.12.3.2. The effects of column widths and beam depths on the rigidity of frames should be evaluated. This is particularly important for concrete frames or for steel frames with relatively deep members and short spans or low story heights.

A8.12.3.3. The effects of the floor slab system acting compositely with the frame beams or girders. Although the composite action may have an insignificant effect in resisting negative moments, it provides a significant contribution to the effective beam moment of inertia for positive moments and increases the stiffness of the beams acting as members of a rigid frame. In most cases, the beams will be modeled as prismatic members and engineering judgment will be required to determine an effective portion of the floor system to be modeled compositely with the beams. This composite action is used in the model to calculate the dynamic characteristics, but should be reevaluated for member design to resist negative moments.

A8.12.3.4. The effects of structural elements that are not included in the lateral-force-resisting system may include flat-slab and column systems and structural steel frames with standard connections. The effects of these elements on the stiffness of a building with shear walls or braced frames may properly be ignored, but they may have a significant effect on the stiffness of a building with a moment frame lateral-force-resisting system. In the latter case, the moment frames will be designed to resist 100 percent of the lateral forces, but the modeled stiffness of the frames will be adjusted to reflect the additional stiffness of the above elements, including any torsional effects due to asymmetry in the location of elements.

A8.12.3.5. The effects of relatively rigid nonstructural elements, such as masonry partitions, will be evaluated. If the stiffness of these elements is significant as compared to the stiffness of the assumed lateral-force-resisting system, the elements will be designed and reinforced as shear walls or will be isolated from the structural system by means of expansion joints at the sides and top of the element.

A8.12.3.6. Evaluate the effects of assumptions for modeling shear walls of various cross-sections; for example, the relative stiffness of an L-shaped wall and a wall that consists of a single plane; and the relative stiffness of a shear wall system and a moment frame system.

A8.12.4. Load Combinations. Members of the structure will have the member strength (MS) to resist the effects of the design load combinations shown in Equations A8-10 and A8-11. A limited number of elements may have load combinations that exceed the MS as specified in paragraph A8.12. Equation A8-10 is used when the gravity loads are in the same sense as the seismic loads (e.g., both sets of loads result in compression in a column or negative bending moments in a beam). Equation A8-11 is generally used when there is a potential for load reversal (e.g., tension in column due to seismic loading may be greater than compression due to minimum dead load, or the positive bending moment due to minimum dead load). The 1.2 and 0.8 coefficients for the dead load are established to represent possible vertical seismic accelerations as well as some uncertainties in the actual dead weight of the structure.

$$MS \geq 1.2D+1.0L+1.0E \qquad \text{(A8-10)}$$

$$MS \geq 0.8D+1.0E \qquad \text{(A8-11)}$$

where:

MS = member strength (as defined in paragraph A8.9.4)

D = dead load

L = live load

E = earthquake

A8.12.6. Vertical Accelerations. The vertical component of earthquake motion (i.e., up and down motion) will be considered in the design of horizontal cantilever and horizontal prestressed elements. For horizontal cantilever elements, these effects will be satisfied by designing for a net upward force of 0.2D as an additional load case. For other horizontal elements employing prestressing, these effects will be satisfied by substituting Equation A8-12 for Equation A8-11, where D represents the member forces due to the vertical dead weight and E represents those due to the horizontal earthquake forces. These provisions parallel those of AFJMAN 32-1049V1 (Reference 7).

$$MS \geq 0.5D + 1.0E \qquad \text{(A8-12)}$$

A8.12.6. Defining EQ-Y. This is the earthquake in which the structure reaches the yield limit and remains essentially or nearly elastic. In other words, EQ-Y is defined by the capacity of the structure whereas EQ-I is defined by probability of occurrence of the demand at the site. If, for a particular structure, EQ-Y is equal to EQ-I, then the structure satisfies the requirements of EQ-I. However, if EQ-Y is greater than EQ-I, the structure is stronger than the minimum requirements. EQ-Y needs to be defined for use in evaluating nonstructural elements in accordance with ETL 97-11, *Mitigation of Non-Structural Seismic and High Wind Deficiencies for Existing Buildings.*

A8.12.7. Floor Accelerations. Floor accelerations are required to evaluate the floor diaphragms and nonstructural items attached to the building. The maximum modal floor accelerations (a_{XM}) are determined from the modal analysis methods described in Attachment 11. Modal floor accelerations can be obtained from Equations A8-13, A8-14, or A8-15, each being variations of equations in Attachment 11, where acceleration is expressed as a ratio of gravity and defined as force divided by weights (i.e., F=ma, m=w/g, and a=F/m = (F/w)g). Maximum story accelerations may be obtained by the square-root-of-sum-of-squares (SRSS) rule for modal combination. Floor accelerations are used to establish criteria for the design of elements attached to the floors of the building. In three-dimensional analyses, should there be an appreciable amount of rotation of the horizontal diaphragms, the accelerations at points of interest at various locations on each floor level will be determined. Modal accelerations at these locations can be calculated from Equation A8-15, using the corresponding modal displacements:

$$a_{XM} = PF_{XM}S_a \qquad \text{(A8-13)}$$

$$a_{XM} = F_{XM}/w_X \tag{A8-14}$$

$$a_{XM} = \delta_{XM}\,(2\pi/T)^2 \div g \tag{A8-15}$$

Considering the fundamental mode only, these equations for evaluation become:

$$a_X = [\Sigma w_i\phi_i/\Sigma w_i\phi_i^2]\phi_x S_a \tag{A8-16}$$

$$a_X = F_X/w_X \tag{A8-17}$$

$$a_X = \delta_x(2\pi/T)^2 \div g \tag{A8-18}$$

A8.13. Analysis for EQ-II. This section prescribes detailed requirements and describes procedures for verification of performance in EQ-II. The structure is required to perform within the limits prescribed in Table 2 of this ETL.

A8.13.1. Simplified Modal Analysis. The EQ-II analysis incorporates the elastic analysis procedure and AFJMAN 32-1049V2, which uses linear-elastic dynamic analysis performance of a structure. The use of these provisions is appropriately simplified for the purposes of evaluation by the use of “single mode” assumptions and approximate stress analysis procedures discussed in Attachment 9.

A8.13.2. Force Displacements. Buildings will be analyzed on the basis of forces and displacements resulting from the application of the EQ-II response spectra representing the return period specified in Table 2, Performance Requirements for Seismic Loads, of this ETL. For Performance Category I, the return period is 1000 years. For Performance Categories III and IV, the return period is 500 years. The appropriate spectral acceleration values for EQ-II, S_{MS} and S_{ML}, are obtained from the table of Attachment 2. Substitution into Equations A8-2 and A8-3 yields the appropriate spectral acceleration coefficients for evaluation purposes.

A8.13.3. Load Combinations. The demands on the structure will be equal to the combined effects of the dead (D), live (L), and seismic (E) loads as shown in Equations A8-19 and A8-20, where D is the actual dead load and L* is equal to a realistic estimate of the actual live load that will be in place of the time of the earthquake. The value of L* may be as low as 25 percent of the design live load (L). This reduced gravity loading is justified on the basis that the probability is low that both maximum live loads and maximum earthquakes will occur at the same time.

$$\text{Demand} = D + L^* + E \tag{A8-19}$$

$$\text{Demand} = D + E \tag{A8-20}$$

A8.13.4. Outline of EQ-II Evaluation Method. This section outlines the methodology to be used in evaluating individual elements for overstress during EQ-II. Basically, the structure is analyzed using approximate analysis methods to calculate the demands on each element for comparison to the capacities (i.e., MS). The demand/capacity ratios (DCRs) are an indication of the ductility that may be required for the structural element to withstand the forces of the EQ-II earthquake. The structure is considered acceptable if the DCRs do not exceed the IDRs (inelastic demand ratios) specified in Table A8.9. The procedure consists of the specific steps stated in the following paragraphs.

Table A8.9. Inelastic Demand Ratios, IDR

Building System	Element	EQ-II Performance Range C_2	EQ-II Performance Range C_3	EQ-III
Steel DMRSF	Beams	2.00	2.50	3.00
	Columns[1]	1.25	1.50	1.75
Braced Frames	Beams	1.50	1.75	2.00
	Columns[1]	1.25	1.50	1.75
	Diag. braces[2]	1.25	1.50	1.50
	K-Braces[3]	1.00	1.25	1.25
	Connections	1.00	1.25	1.25
Tie Rods	Tension only	1.00	1.10	1.75
Concrete DMRSF	Beams	2.00	2.50	3.00
	Columns[1]	1.25	1.50	1.75
Concrete Walls:				
(1) Single Curtain of Reinforcing	Shear	1.10	1.25	1.50
	Flexure	1.50	1.75	2.00
(2) Double Curtain of Reinforcing	Shear	1.25	1.50	1.75
	Flexure	2.00	2.50	3.00
Diaphragms	Shear	1.25	1.50	1.75
	Flexure	1.50	1.75	2.00
Masonry Walls	Shear	1.10	1.25	1.50
	Flexure	1.50	1.75	2.00
Wood	Trusses	1.50	1.75	2.00
	Columns[1]	1.25	1.50	1.75
	Shear walls	2.00	2.50	3.00
	Connections (other than nails)	1.25	1.50	2.00
Concrete Frames	Beams	1.25	1.50	1.75
	Columns[1]	1.00	1.25	1.25
Unreinforced Concrete Walls	Shear	1.00	1.10	1.25
	Flexure	1.00	1.00	1.00
Unreinforced Masonry Walls	Shear	1.00	1.10	1.25
	Flexure[4]	1.00	1.00	1.00

[1]In no case will axial loads exceed the elastic buckling capacity.
[2]Full panel diagonal braces with equal number acting in tension and compression for applied lateral loads.
[3]K-bracing and other concentric bracing systems that depend on compression diagonal to provide vertical reaction for tension diagonal.
[4]Includes in-plane (rocking shear) and out-of-plane.

A8.13.4.1. Form the response spectrum for EQ-II using Equations A8-1, A8-2 and A8-3 and the spectral acceleration coefficients from Attachment 2. The response spectrum will be that specified for EQ-II with damping as specified in Table A8.6 for the elastic analysis procedure.

A8.13.4.2. Perform a simplified modal analysis of the structure using approximate analysis methods of Attachment 9 and the EQ-II response spectrum.

A8.13.4.3. Calculate the forces on all of the structural elements using the load combinations of Equations A8-19 and A8-20. These forces will be defined as the demand forces and denoted with subscript D (e.g., M_D, V_D, F_D). The simplified form of modal analysis (e.g., single mode) will be used unless denied by structural irregularity, number of degrees of freedom, or complexity.

A8.13.4.4. Calculate the yield or plastic strengths of all the structural members, using the same force units used for demands. These forces will be defined as the capacity forces and denoted with the subscript C (e.g., M_C, V_C, F_C).

A8.13.4.5. Calculate the ratio of the demand force to the capacity force for each of the structural elements. These ratios are the demand/capacity ratios. A graphical illustration for flexible members is shown in Figure A8.4. A method for determining the DCRs for steel and reinforced concrete columns is shown in Figures A8.5 and A8.6, respectively. The equations in these figures were adapted from the general interaction equations for steel and concrete (Equations A8.1-1 and A8.1-1b, AISC LRFD Specifications). The member strength of infill masonry wall panels can be approximated as described in *Seismic Design for Essential Buildings* (Reference 6).

A8.13.4.6. Review the DCRs for uniformity, symmetry, mechanisms, and relative values. The DCR values are an indication of the ductility that may be required for the structural element to withstand the forces of the criteria earthquake. Compare the DCR values with the Inelastic Demand Ratio (IDR) limits provided in Table A8.9.

- Exceeding the Inelastic Demand Ratios (IDRs) of Table A8.9.
- Asymmetrical yielding, on a horizontal plane, that will decrease the torsional resistance.
- Hinging of columns at a single story level that will cause a mechanism.
- Discontinuity in vertical elements that can cause instability or fracture.
- Unusual distributions of Inelastic Demand Ratios.

A8.13.4.7. Engineering judgment is required for this structural evaluation of the post-yield analysis. If the review of the Inelastic Demand Ratios satisfies the requirements of paragraph A8.13.4.6 above, it may be assumed that the inelastic drift is adequately approximated by the elastic analysis. If the review does not satisfy the requirements and the elastic analysis method is judged unacceptable, the building will be reevaluated using the capacity spectrum method.

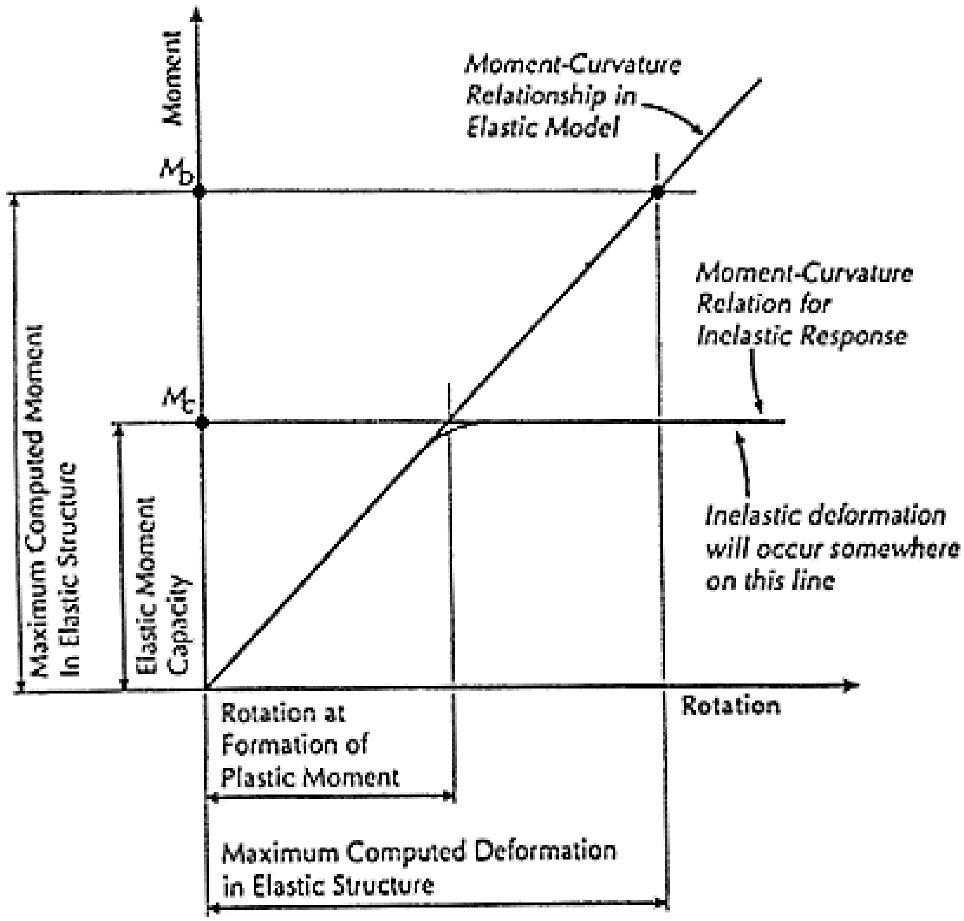


Figure A8.4. Definition of Inelastic Ratios for Flexural Members

1. At a Braced Location:

$$\frac{M_x}{M_{pcx}} + \frac{M_y}{M_{pcy}} = DCR \le IDR$$

2. Stability Between Braced Points:

$$\frac{C_{mx} M_x}{M_{ucx}} + \frac{C_{my} M_y}{M_{ucy}} = DCR \le IDR$$

where:

P, M_x, and M_y	=	axial load and moments from first order elastic analysis
M_{pcx}	=	$1.18M_{px}$ [1-(P/P_y)]
M_{pcy}	=	$1.19M_{py}$ [1-(P/P_y)^2]
M_{ucx}	=	M_{ux} [1-P/P_{cr})][1-(P/P_{ex})]
M_{ucy}	=	M_{py} [1-(P/P_{cr})][1-(P/P_{ey})]
M_{px}, M_{py}	=	plastic moment capacities

$$M_{ux} = M_{px}\left[1.07 - \frac{L / r_y \sqrt{F_y}}{3160}\right] \le M_{px}$$

P_{ex}, P_{ey}	=	Euler buckling loads for x and y axes
P_{cr}	=	$1.7AF_a$(P/$P_{cr} \le 0.5$)
C_{mx}, C_{my}	=	$0.6 - 0.4(M_1/M_2) \ge 0.4$
IDR	=	allowable ductility (inelastic demand ratio)
DCR	=	demand capacity ratio

Figure A8.5. Ductility Check of Steel Columns

1. Compression:

$$\frac{M_x}{M_{ux}} \frac{1-b}{b} + \frac{M_y}{M_{uy}} = DCR \leq IDR$$

$$\frac{M_y}{M_{uy}} \frac{1-b}{b} + \frac{M_x}{M_{ux}} = DCR \leq IDR$$

2. Tension:

$$\frac{M_x}{M_{mx}} \frac{1-b}{b} + \frac{T}{T_u} = DCR \leq IDR$$

$$\frac{M_y}{M_{my}} \frac{1-b}{b} + \frac{M_x}{M_{mx}} + \frac{T}{T_u} = DCR \leq IDR$$

$$\frac{T}{T_u} < 0.5$$

where:

M_x, M_y, and T	=	Moments and net axial tension from elastic analysis
M_{ux} and M_{uy}	=	Uniaxial ultimate moment capacities from interacting diagrams
M_{mx} and M_{my}	=	Uniaxial ultimate moment capacities in the absence of axial load
T_u	=	Ultimate tensile capacity of vertical reinforcement = $\Sigma A_s F_y$
β	=	Coefficient from PCA Advanced Engineering Bulletin No. 20 (Reference 33)
IDR	=	Allowable ductility (inelastic demand ratio)
DCR	=	Demand capacity ratio

Figure A8.6. Ductility Check for Concrete Columns.

A8.13.4.7.1. The capacity spectrum method is described in AFJMAN 32-1049V2, *Seismic Design Guidelines for Essential Buildings*. It is an approximate inelastic analysis procedure. The ability of the building to resist the forces and deformations caused by an EQ-II or EQ-III event is determined by a graphical method. Two curves are constructed. One curve represents the capacity of the structure to resist the lateral forces. The other curve represents the demand of the ground shaking. The capacity curve is developed from a force (F or V) versus displacement (δ) relationship of the overall structure. Modal analyses are used to determine levels of excitation to yield structural elements. The capacity is defined by the forces and displacements of the fundamental mode. The force-displacement curve can be converted into a spectral acceleration (S_a) versus period (T) curve (i.e., a capacity spectrum) by means of the standard equations for modal story lateral forces, modal base shear, and modal deflections and drift. The demand of the ground shaking is represented by either an EQ-II or EQ-III response spectrum curve. The capacity and demand curve are plotted on the same graph; their intersection is considered to be the reconciliation between demand and capacity. The following is a step-by-step outline description of the capacity spectrum method for approximating the inelastic capacity of the structure:

1. By use of a modal analysis, determine the level of excitation that causes first major yielding of the structure.
2. Revise the stiffness or resistance characteristics of all structural elements that are within 10% of their yield capacities to represent a plastic hinge.
3. Apply additional lateral forces to the structure, by means of a modal analysis, until an additional group of structural elements reaches their yield capacities.
4. Repeat the above until the combined results reach an ultimate limit (e.g., a mechanism, instability, or excessive distortions).
5. Convert the results into a capacity curve based on the periods and spectral accelerations for the fundamental mode of vibration.
6. Graphically compare the demand of the evaluation EQ response spectrum to the capacity of the structure.
7. Approximate the lateral deformations and compare to the drift limits given above.

A8.13.5. Guidance for Review of DCRs.

A8.13.5.1. Asymmetrical Yielding on a Horizontal Plane. This provision is used to check for the possibility of torsional instability (paragraph A8.9.3). For example, if all the DCRs on the north side of the structure were greater than 1.0, and all the ratios on the south side were less than 1.0, a potential for torsional instability exists. Yielding of the north side will reduce the stiffness of that side of the building relative to the south side, thus the center of rigidity moves to the south. If this condition increases the horizontal eccentricity of the building, torsional moments increase geometrically and the potential for collapse is present.

A8.13.5.2. Hinging of Columns at a Single Story. This provision is used to check for the possibility of an unstable soft story. For example, if DCRs were equal to about 1.5 at the tops and bottoms of 80 percent of the columns for the first story of a multistory building and DCRs for columns at every other story were less than 1.0, the potential for instability at the first story exists. Because the columns are yielding only at the first story, all the inelastic energy will have to be absorbed at that level. This subjects the first story to the possibility of excessive interstory displacements.

A8.13.5.3. Unusual Distributions of DCRs. This is a more general case of paragraphs A8.13.5.1 and A8.13.5.2. This provision is used to check the efficiency of the overall lateral-force-resisting system. If a limited number of structural elements have large DCRs and the remainder of the elements have ratios less than 1.0, it might be prudent to consider some structural modifications to reduce the potentially high demands on a small number of structural elements.

A8.13.6. Approximate Strength of Masonry In-Fill Panels. This section offers some guidance on how concrete masonry infill walls may be approximated for evaluation purposes.

A8.13.6.1. Elements Yielded in In-Plane Flexure. The in-plane flexural capacity of the infill wall can be approximated by the following usual equations where the terms are as identified in Figure A8.7:

$$M_n = A_s f_y (d - a/2) \qquad \text{(A8-21)}$$

or

$$M_n = 0.85 f_m' ab(d - a/2) \qquad \text{(A8-22)}$$

where:

$$a = A_s f_y / 0.85 f_m' b = 0.85c \qquad \text{(A8-23)}$$

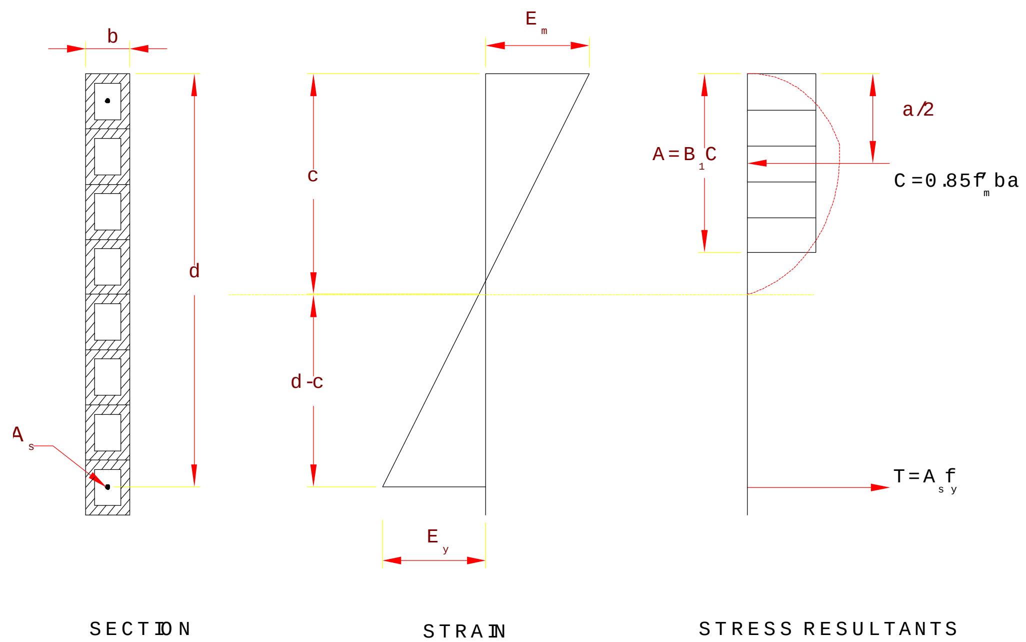


Figure A8.7. Force and Strain Diagrams

A8.13.6.2. Elements Yielded in In-Plane Shear. In treating a masonry panel yielded in in-plane shear, consider the panel as an infill panel within a frame. In this approximation, the vertically grouted reinforced edge cells and the upper and lower floors or beams serve as boundary elements for the frame. Two failure modes (Reference 26) are adopted for evaluation. First, diagonal tensile cracking of the masonry panels leads to the development of a diagonal strut as illustrated in Figure A8.8a. The structural model is changed by replacing the panel with the equivalent diagonal brace. Second, sliding shear failure can occur. If it does, the equivalent structural mechanism changes from the diagonal braced pin-jointed frame to the knee-braced frame shown in Figure A8.8b. In considering both failure modes, the equivalent diagonal compressive strut is defined and placed in position as a replacement for the panel. The diagonal compression failure force, R_c, of the strut is calculated. The equivalent diagonal strut compression force, R_s, to initiate horizontal shear sliding is also calculated. If either failure force is reached or exceeded, the infill panel is considered completely failed and of no further structural value. The dimensions of the equivalent diagonal strut and failure forces can be calculated using the following equations.

A8.13.6.3. The diagonal strut capacity considering compression failure is:

$$R_c \quad = \quad 0.667ztf'_m\sec\theta$$

where:

$$Z = \frac{p}{2}\left[\frac{\Delta^3}{3\sin 2q}\right]^{1/4} \leq 0.125\frac{d_m}{\sin q}$$

θ = angle between strut and horizontal as shown

Δ = length of grouted edge cell

d_m = length of equivalent diagonal strut

f'_m = masonry compressive strength

t = effective wall thickness

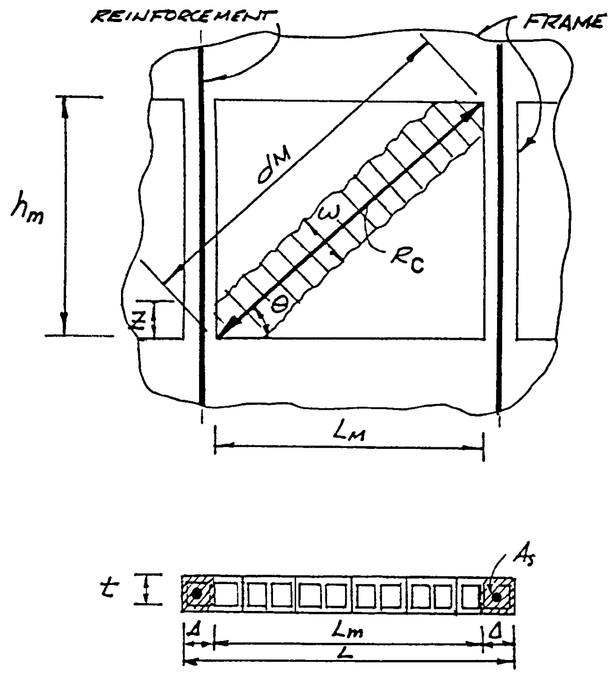


Figure A8.8a. Diagonal Tension Mode

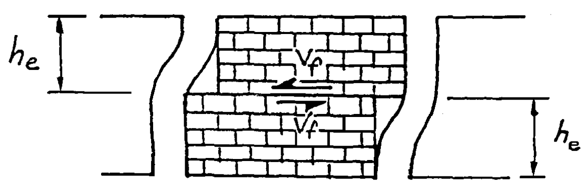


Figure A8.8b. Sliding Shear Failure Mode

A8.13.6.4. The diagonal strut capacity considering sliding shear failure is:

$$R_S = \frac{\left(0.03 f_m' + 0.3 f_m\right)}{1 - 0.3\left(\frac{h}{L}\right)} d_m t$$

where:

t = panel thickness

h = panel height

L = panel width

f_m = allowable masonry compressive strength

A8.13.6.5. In situations involving regular frame and infill panel construction where the infill panel is placed after the boundary members (e.g., columns, beams, and floor slabs), the $0.3f_m$ term in the numerator is taken as zero since there is no significant transverse compressive stress. A horizontal space normally exists between the top of the infill panel and the overhead beam or slab boundary member.

A8.13.6.6. Additional Modeling Considerations. Additional modeling information for members and structures of masonry and other materials is included in FEMA 273 (Provisions and Commentary) (Reference 13).

A8.13.7. Modal Analysis. The simplified modal analysis follows the procedures of Attachment 9 as modified in paragraph A8.10.

A8.13.8. EQ-II Mathematical Modeling. The comments of paragraph A8.12.3 regarding EQ-I analysis modeling generally apply; however, some modification to the modeling assumptions may be made.

A8.13.8.1. Allowances may be made to account for the reduced section properties of cracked or partially cracked concrete.

A8.13.8.2. Allowances may be made for flexibility at beam-column joints.

A8.13.8.3. Unless the floor slab system is integrated into the design of the beams and girders, composite action need not be considered.

A8.13.8.4. The effects of nonseismic frames should be reevaluated with regard to the larger deformations resulting from EQ-II. These effects would usually be ignored in the mathematical model unless they provide redundancy for the overall lateral-force-resisting system.

A8.13.8.5. The effects of nonstructural elements is not included in the mathematical model to calculate periods, displacements, and member forces. However, the possible detrimental effects of rigid nonstructural elements must be considered in the overall evaluation of the structure.

A8.13.8.6. The modification of modeling assumptions can result in the lengthening of periods of vibration by 25 percent to 50 percent.

A8.14. Analysis for EQ-III. An evaluation for EQ-III uses the same procedure for EQ-II evaluation. The applicable spectral acceleration values S_{MS} and S_{ML} for EQ-III are obtained from the Table A2.1 of Attachment 2. Substitution into Equations A8-2 and A8-3 yields the appropriate spectral acceleration coefficients for evaluation purposes. IDR values, allowable drift limits and damping values are provided in the same tables used to acquire the corresponding values for EQ-II analysis. A check of DCR values must still be made for EQ-III if the NDFEF exceeds 1.5, even if the structure satisfies the EQ-II criteria.

A8.15. Computer Programs. Although the use of a computer program will generally be more efficient and give more accurate results, the single-mode analysis prescribed for most evaluations under this ETL can be done by hand calculations or hand calculator with adequate accuracy and reasonable effort. Alternately, the evaluator may use nationally recognized commercial computer software. Two or three dimensional computer programs are discussed in paragraph A11.10 of Attachment 11.