COMMENTARY ON CHAPTER 5
EARTHQUAKE HAZARD REDUCTION IN EXISTING CONCRETE BUILDINGS AND CONCRETE WITH MASONRY INFILL BUILDINGS

23 January , 2003

Preface

Concrete frame buildings have been recognized as being susceptible to damage from seismic activity. The San Fernando Earthquake of 1971, the Mexico City Earthquake of 1985, the Whittier Narrows Earthquake of 1987, the Loma Prieta Earthquake of 1989, the Northridge Earthquake of 1994, the Kobe Earthquake of 1995, the Izmit (Turkey) Earthquake of 1999, and the Gi-Gi (Taiwain) Earthquake of 1999 caused substantial damage to concrete frame buildings.

Development of this chapter was derived from several sources. FEMA and the ATC developed a document (called ATC-33, later known as a Prestandard FEMA-356} that developed a methodology to analyze existing buildings. Included in this document were provisions for analysis of concrete buildings. ATC-40 was another document intended to assist engineers specifically in the analysis of concrete buildings. In response to Senate Bill 547, the City of Los Angeles took steps to develop provisions (Division 95) to be utilized for concrete frame buildings within the City's jurisdiction. SEAOC has had ongoing activity relating to the analysis of non-ductile concrete buildings. Also, the Strong Motion Instrumentation Program (SMIP) has funded study relating to non-ductile concrete buildings.

The loss of lives and property in future earthquake will be heaviest in buildings that exist today. The short-term impact of improved knowledge, codes, and design practices will be limited to those relatively few recent buildings that take advantage of this improved knowledge. Thus the dominant policy issues posed by earthquakes involve not new but existing buildings, particularly those structures that have obvious weaknesses and do not comply with the general intent and necessary requirements of current regulations. The issue before the public and the professions is how to set standards for these non-compliant existing buildings that are consistent with both the desire for safety and the limited resources available to achieve improved safety.

It is generally accepted that the intensity of earthquake which could be reasonably be expected to occur in California would be sufficient to cause buildings with minimal seismic resistance characteristics to be seriously damaged or, perhaps, to collapse, causing serious injury or death to the occupants or passers-by.

It is reasonable, when a real hazard exists, to take steps to significantly reduce the hazard. The objective of Appendix Chapter 5, is the reduction or mitigatation of hazard to the greatest extent practicable. Application of these Provisions will decrease the probability of loss of life, but loss of life cannot be prevented. Also, we should be willing to accept some major and irreparable damage as long as there is a decrease in the likelihood of loss of support for horizontal framing.

The goal of Appendix Chapter 5 lower than the goal set for new construction. It should be recognized that the economic difficulty of strengthening existing buildings necessitates reliance on building components with seismic performance characteristics that are less than ideal.

C- 501 –PURP0SE

No commentary is provided for this section

C- 502 - SCOPE

The concept of "Benchmark Building" used in ASCE 31-02 "*Seismic Evaluation of Existing Buildings"* is utilized to limit the scope of this appendix chapter. While benchmark buildings need not proceed with further evaluation, the design professional must clearly demonstrate the building is compliant with the benchmark document. One exception to the use of benchmark buildings is if the seismicity of the region has changed since the design of the building.

Exception No. 2 exempts this type of building because the semi-flexible or flexible roof diaphragm modifies the seismic response from that anticipated by this Appendix. This exemption should not considered as an assumption that these buildings may not be hazardous. The analyst should use a structural model that includes the appropriate stiffness of the roof diaphragm.

Exception No.3 is a committee decision that the seismic hazard in this geographic region does not warrant an analysis nor retrofit.

C- 503 – DEFINITION

No commentary is provided for this section.

C- 504 – SYMBOLS AND NOTATIONS

No commentary is provided for this section.

C- 505 – GENERAL REQUIREMENTS

C – 505.1 General

This chapter provides a 3-tiered procedure to evaluate the need for seismic rehabilitation of existing concrete buildings and concrete buildings with masonry infills. This chapter does not preclude a building from being evaluated or rehabilitated to conform the acceptance criteria using other well established procedures, such as ASCE 31-02, a prestandard FEMA 356, *"Seismic Rehabilitation of Buildings"* or ATC 40, "*Seismic Evaluation and Retrofit of Concrete Buildings"*. Approval of the authority having jurisdiction shall be obtained before using alternative procedures.

Concrete buildings with masonry infill should have special consideration of the seismic response of this class of buildings. It is not conservative to ignore the contribution of masonry infills to the structural stiffness. These infills may cause the building to have weak and/or soft stories or significant torsional response. Psuedo-nonlinear modeling of the infill, using the testing results obtained as specified in Section 510, in a Tier 3 procedure, includes the effective stiffness of the infills in the dynamic analysis.

C- 505.2 Properties of in-place materials.

The stress-strain relationship of existing concrete, masonry and reinforcement shall be determined from published data or by testing. Section 505.2.1 provides exceptions for Tier 1 and Tier 2 analyses.

All available information from the sources given in this Section shall be considered when determining material properties.

C- 505.2.1 Concrete.

The compressive strength of cores taken shall be determined by testing for Tier 3 analyses. The engineer may use core testing for determination of concrete compressive strength for Tier 1 and 2 analyses. Cutting of cores shall be carefully planned by the engineer to not significantly reduce the shear or flexural strength of the existing structural system. Cutting of any existing reinforcement is prohibited. The engineer will have to prepare in writing a coring program and provide an on-site representative to enforce this mandate. Cutting cores in columns is also prohibited. The total area of a column has an existing axial stress. Patching of a core hole in a column will not restore of the column because the patching material is installed at near zero stress and will be subject to shrinkage.

This Section allows the compressive test results obtained from testing in areas other than columns to be assumed for the columns. If the construction documents specify a higher compressive strength for columns than that of beams and shear walls, the higher strength specified for the columns may be used in the analysis if the testing shows that the strength specified for beams is verified by the cores taken from the beams.

The mean value of the compressive strengths obtained from the test program shall be used in the analyses.

C- 505.2.2 Solid grouted reinforced masonry.

The engineer may use the prism compressive strength specified in this Section or may elect to cut prisms from the existing infill walls and test in accordance with the procedures given in the Building Code. The strain associated with peak stress shall be determined by prism testing or taken as 0.0025 inches per inch.

The size and number of prisms cut shall conform to the requirements of the Building Code for testing of constructed masonry. Prisms should not be cut from the areas of high stress in infills. These areas are adjacent to the intersections of beams and columns and adjacent to corners of wall openings.

C- 505.2.3 Partially grouted masonry.

The prism compressive strength of partially grouted masonry may be determined by methods given in the Building Code as *"Unit Strength Method"*. Units are taken from the wall by removing mortar joints for unit testing. The number of units needed for testing shall conform to the Building Code. The ratio of the number of unit strength method test areas to wall area should conform to that specified in the Building Code *"Testing prisms from constructed masonry"*. The assumed strength of the grout for the unit strength method shall not be assumed as less than 2000 psi or a higher compressive strength of the grout may be determined by testing grout cores. Mortar need not be analyzed for cementitious materials. Mortar may be assumed to be Type S.

C- 505.2.4 Unreinforced masonry.

The procedure given in this Section is intended for solid masonry, either clay or concrete. The "flat jack" method given in Section 510 cannot be used in cored brick or in hollow core unit masonry. It is recommended that the compressive strength of cored or hollow unit masonry be determined as specified in Section 505.2.3.

C- 505.2.5 Reinforcement.

The expected yield stress of new or existing reinforcement shall be taken from Table 505.2 unless sampled and tested for yield stress. Table 505.2 is a tabulation of expected yield stress, not a minimum yield stress. If tested yield stress values exceed those shown in Table 505.2, the mean of the test values should be used.

Underestimation of reinforcement yield stress is not a conservative assumption. The limit shear loading of a beam, column or wall is determined in accordance with Section 509.1.2.3. The probable moments should be based on expected yield moments. The concrete strain at yield rotation is determined by the sum of axial loading and yield stress of all reinforcement within the cracked concrete section. Again, understatement of probable reinforcement yield stress is non-conservative.

C- 506 - SITE GROUND MOTION

C- 506.1 Site ground motion for Tier 1 analysis.

The ground motion [siesmic base shear] used for a Tier 1 analysis is the same as used for the design of a new building. The concept is that finding an existing concrete building is in conformance with a procedure that involved a minimum of inspection and evaluation should have a lower probability of occurrence than a more rigorous procedure.

C- 506.2 Site ground motion for Tier 2 analysis.

The concept is that the purpose of this Chapter is earthquake hazard reduction. This does not imply that existing buildings must fully conform to the standards for new construction for reduction of hazard. This reduction of the seismic loading is identical to FEMA- 178 /June 1992 *"NEHRP Handbook for the Seismic Evaluation of Existing Buildings"*.

C-506.3 Site ground motion for Tier 3 analysis.

The loading criteria for a Tier 3 analysis is an elastic design response spectrum compatible with mapped spectral acceleration and velocity parameters as modified by soil site class factors. The spectral acceleration values shall be reduced as permitted for a Tier 2 analysis. The spectral acceleration values are increased by occupancy importance factors when applicable.

All Tier 3 analysis procedures are displacement based. The building is displaced relative to its base in appropriate mode shapes in its linear and nonlinear response. The acceptance criteria is a limit on material strain, global strength degradation and shear strength..

C- 507 - TIER 1 ANALYSIS PROCEDURE

This section provides a means for conducting a simplified and rapid analysis evaluation of existing concrete buildings. This section is not intended to be used for concrete buildings with masonry infill.

The evaluation procedure is patterned after ASCE 31-02 *"Seismic Evaluation of Existing Buildings"* with emphasis on Tier 1 Analysis for Building Type C1: Concrete Moment Frame, and Type C2: Concrete Shear Walls. Nevertheless, the analysis is based on an equivalent lateral force procedure in lieu of the displacement method used by the referenced document.

ASCE's regions of seismicity and performance level criteria have been converted into two seismic hazards categories to closely approximate the occupancy categories of UBC 97 and IBC 2000.

C-507.2 Limits.

In an effort to maintain the simplicity and rapid nature of the evaluation procedure, this section is limited to regular buildings.

The visual inspection required verifying the regularity of the building should include both an on-site inspection of the building and review of the record drawings. When record drawings are not available, some forms of non-destructive inspection may be required.

C-507.3 Evaluation Report.

The evaluation report is intended to serve as a summary of the evaluation performed for review by the building official. It should clearly indicate one of two things: a) No Non-compliant items exist in the building and as such no rehabilitation measures are necessary, or b) Non-Compliant items do exist in the building and the subsequent step followed by the design professional in addressing these items. As noted in Section 807.4, the design professional has the option of performing subsequent analysis (Tier 2 or Tier 3) to show the Non-Compliant items are acceptable or pursue a mitigation effort to correct the deficiencies noted per Tier 1.

When a subsequent analysis is performed, the report required under this section is intended to be used as supporting documentation for limiting the analysis to only the Non-Compliant Items.

C-507.4 Evaluation Procedure

The design professional may seek to address non-compliant items directly within a Tier 1 analysis but it must be based on an acceptable rational analysis acceptable to the building official. Since such analysis does exist under Tier 2 or 3, the engineer or architect is encouraged to address non-compliant items accordingly.

C-507.6 Quick Check Analysis.

The lateral force level used for performing the quick check analysis, or answering evaluation statements, are based on applying 100% of the Building Code force level.

In computing the lateral force, the design professional must select an appropriate R-value. In making a determination, the design professional should consider the buildings confinement detailing, redundancy, toughness, overstrength, etc. As a guide, one may use the Building Code in determining the appropriate R-value to use.

C-508 TIER 2 ANALYSIS PROCEDURE

C-508.1 General.

The Tier 2 linear analysis procedure is a building code type of analysis procedure, using a single "global" response modification factor R for the entire structure. This approach differs from the linear analysis procedure used in FEMA 356, which employs a “component based” ductility related factor “m” in checking the acceptability of the component evaluated.

The R factor shall be selected based on the type of seismic-force-resisting system employed, e.g., for Ordinary Reinforced Concrete Moment Frame, uses R=3; for Ordinary Reinforced Concrete Shear Walls, use R=5; for Detailed Plain Concrete Shear Wall uses R=3. The designer shall refer to the Building Code, Chapter 16 for a complete list of Seismic-Force-Resisting system and associated design coefficients.

C-508.2 Limitations.

Limits of Linear procedure are given. Linear procedures are most applicable to buildings that actually have sufficient strength to remain nearly elastic when subjected to the design earthquake, or buildings with regular geometries and distributions of stiffness and mass.

For buildings with plan irregularities or weak stories, the inelastic ductility demand may significantly differ from the result of linear analysis, therefore, Tier 3 procedures shall be used. For buildings with vertical irregularities or significant higher mode response, Linear Dynamic procedure may provide a more accurate distribution of seismic demand as compare to the Linear Static procedure.

C-508.3 Analysis Procedure.

The stiffness of the component shall be calculated using the values given in Table 808.1. The building period calculated using the effective stiffness shall not be more than 1.3 times the method A period. For determining the effective width of a flat plate or flat slab, the designer shall refer to the available research data.

C-509-TIER 3 ANALYSIS PROCEDURE

C-509.1 General.

This procedure is called a psuedo-nonlinear dynamic analysis because it is an iterative procedure that modifies the stiffness matrix for each subsequent iteration. The effective stiffness used in the iterative procedure is the element or system stiffness is taken from a nonlinear force-displacement of that element or system as a linear secant stiffness.

C-509.1.1 Mathematical Model.

The mathematical model must represent the existing building as discovered in the survey. All building elements that extend the full story height in any story or restrict the curvature of any element must be included in the model.

Cast-in-place reinforced floor diaphragms that have a span-depth ratio of less than that specified may be assumed to be rigid elements in the model. Floor diaphragms constructed of precast elements, either with or without cast-in-place reinforced concrete topping, shall be analyzed to determine if they must be considered as semi-rigid diaphragms. Semi-rigid diaphragms must be represented in the mathematical model with a cracked (effective) stiffness. The probable elastic response of the diaphragm to the seismic loading should be used to estimate the effective stiffness.

Parking structures that have diaphragms that ramp between story heights shall have their tributary and self-weight distributed along their span length. The span of the diaphragm may be taken as the distance between the level floors that are at the story levels used in the mathematical model.

C- 509.1.2 Acceptance criteria.

This Section has three separate limits for acceptance. They are maximum values of compressive strain, story drift, and shear strength. The lesser of the limits shall be used for acceptance.

C- 509.1.2.1 Compressive strain determination.

Compressive strain in reinforced concrete elements may be calculated from nonlinear analyses that include warping of originally plane sections or from simplified analyses that assume plane sections remain plane.

The critical strains shall determined from the unreduced element loading calculated by the dynamic response of the mathematical model. The loading of the mathematical model shall be the load combinations of the Building Code for that described as Strength Design or Load Resistance Factored Design.

C- 509.1.2.2 Story drift limitation.

This Section has four separate limits on story drift. The story drift that causes the lesser of the four limits shall be used for acceptance. The limitations are as follows.

A story drift that causes a compressive strain of 0.003 in. per inch is a limit for shear walls and concrete frames that confine masonry infills.These strains are considered acceptable for unconfined concrete. The maximum strain for semi-confined (shell) concrete, that concrete outside the confinement reinforcement, is limited to 0.004 in. per inch. The peak strain in masonry infills is limited to that strain shown acceptable by published experimental data or by the prescribed physical testing of the infill masonry.

The relative displacement at any story level is limited to the strength level displacement in that story. The strength level displacement is defined as the displacement when strength degradation begins in the nonlinear force-displacement relationship of a line of resistance in any story level. This limitation is modified by the exception that permits the beginning of strength degradation to be defined as a reduction of strength on a line of resistance, which is comprised of several elements, of ten percent of the peak strength.

C-509.1.2.3 Shear strength limitation.

The shear strength of columns, piers and shear walls must exceed that calculated from the flexural strength of these elements. No strength reduction factors shall be used in the determination of flexural strengths. The nonlinear force-displacement relationship of an infilled frame panel is determined by an analysis that must include a failure mode. This failure mode can be either compressive strain or shear failure. This analysis complies with this shear strength limitation.

C- 509.2 Psuedo-nonlinear dynamic analysis procedure.

This procedure requires a three dimensional mathematical model of the building. This 3-D model is analyzed for concurrent seismic excitation on orthogonal axes of the building. The demand effects for the concurrent loading are combined by SRSS methods. This 3-D dynamic analysis procedure combines concurrent translational and torsional effects and does not require that torsional effects be magnified by accidental torsion or dynamic amplification of torsion.

C- 509.2.1 Determination of effective stiffness.

This Section provides two methods for determination of the effective stiffness of reinforced concrete elements and of infilled frames. The psuedo-nonlinear dynamic analysis procedure uses the current effective stiffness of elements of the 3-D mathematical model to predict the displacements in the next iteration of a psuedo-nonlinear analysis. The "current effective stiffness" is a secant stiffness estimated from the force-displacement relationship obtained from a nonlinear analysis of each element of a story height on a line of resistance. The iterative procedure concludes when the effective stiffness used is appropriate for the displacement calculated by the last dynamic analysis.

C- 509.2.1.1 General.

The effective stiffness of concrete and masonry elements ,except for infills which are covered in Section 509.2.1.2, is determined by a nonlinear analysis with consideration for the effects of tensile cracking of concrete and masonry and of the reduction of the chord compressive modulus of elasticity due to strain. The force-displacement relationship of elements shall include a determination of the relative displacement at the strength limit state. A force-displacement relationship is determined for the system or element. Judgement for using a primary mode shape or a multi-mode shape shall be used. The calculated story forces will provide guidance for the distribution of loading used in the nonlinear analysis. An exception to the requirement for a nonlinear analysis is provided for beams and columns in shear wall and infilled frame buildings. It is assumed that the shear walls or infilled frames will provide the principal resistance to seismic loading. The initial estimates used in the analyses must be revised if the effective stiffness calculated from the moments determined by the final analysis by more than twenty percent.

C- 509.2.1.2 Effective stiffness of infills.

The effective stiffness of infills is determined by nonlinear analyses of representative infill panels. Representative panels are based on height-length ratios and location of openings (or no openings) within the panel. The confining frame without any infill is also analyzed. The stiffening effect of the infill is the difference between the force-displacement relationships of the frame with infill and without infill.

C- 509.2.1.3 Model of the infill.

The stiffening effects of the infill is modeled as a pair of isotropic diagonals. This simplification is for symmetry in the model for fully reversing cyclical loading. Experimental testing of infilled reinforced concrete frames "*ERDC/CERL TR-02-1, Ghassan Al-Chaar"* shows that using only a portion of the infill as a brace within the frames may provide a strength comparable to the experimental model but will very significantly over-estimate the displacement for each increment of loading.

C- 509.2.2 Description of analysis procedure.

Each iterative step of the psuedo-nonlinear dynamic analysis is identical to a conventional elastic response spectrum except the effective stiffness of elements is used in lieu of an elastic stiffness. The initial estimate of an effective stiffness could be yield stiffness for elements that are likely to have an inplane capacity in excess of probable demand. The iterative procedure is deemed closed when the stiffness used in the current analysis does not deviate by more than 10 percent from that determined from the force-displacement relationship. This closure criteria is for all elements of the structural model.

C- 509.2.2.1 Number of modes.

No commentary

C- 509.2.2.2 Combining modes.

No commentary

C-509.3.1 General.

The Capacity Spectrum Analysis Procedure is contained in ATC-40, "Seismic Evaluation and Retrofit of Concrete Buildings".

Static inelastic methods and dynamic elastic methods are not able to adequately represent the full effect of torsional response. Response amplitudes associated with inelastic torsion may be much larger than those indicated by these approaches. For structures influenced by inelastic torsion, it often is more appropriate to use simple models or procedures to identify approximately the effect of the irregularity on torsional response, and to apply this effect independently to either a two- or three-dimensional static inelastic analysis of the building. Available research may provide insight into the required analysis process (Goel and Chopra 1991; Sedarat and Bertero 1990; Otani and Li 1984). Where inelastic response is expected to be a dominant feature of the overall response, it usually is preferable to engineer a retrofit strategy that reduces the torsional response, rather than try to engineer an analysis procedure to represent inelastic torsion. Also, See Commentary for Section 508.2.3

Soil-structure interaction refers to response modification because of interaction effects, which could include reduction or increase the roof displacement, and modeling of the foundation soil-superstructure system.

Soil flexibility results in period elongation and damping increase. In the context of inelastic static analysis as described in this methodology, the main relevant impacts of soils-structure interaction are to provide additional flexibility at the base level that may relieve inelastic deformation demands in the superstructure. Because the net effect is not readily assessed before carrying out the detailed analysis, it is recommended that foundation flexibility be included routinely in the analysis model.

C-509.3.2.1 Component initial stiffness.

The stiffness values provided in Table 808.1 represent values expected for typical proportions and reinforcement ratios. Some adjustment up or down depending on the actual proportions and reinforcement ratios is acceptable.

C-509.3.2.2 Component strength.

No commentary

C-509.3.2.3 Component deformability.

No commentary

C- 509.3.3 Description of analysis procedures.

C-509.3.3.1 Determination of the capacity curve.

The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This is generally valid for buildings with fundamental periods of vibration up to about one second. For more flexible buildings with a fundamental period greater than one second, the analyst should consider addressing higher mode effects in the analysis.

C-509.3.3.2 Conversion of the capacity to the capacity spectrum.

In order to utilize the Capacity Spectrum Analysis Procedure, both the demand response spectra and the capacity curve need to be plotted in the spectral acceleration vs. spectral displacement domain. Spectra plotted in this format re known as Acceleration-Displacement Response Spectra (ADRS) after Mahaney, 1993.

C-509.3.3.3

No commentary

C-509.3.3.4 Development of the demand spectrum.

In the ATC-40 document, the equations for SR_A and SR_V involve a factor □(kappa). This k factor depends on the structural behavior of the building, which in turn depends on the quality of the seismic resisting system and the duration of ground shaking. There are three categories of behavior for quantification of the k factor in the ATC-40 document. One category of behavior represents stable, reasonably full hysteresis loops, another category of behavior represents a moderate reduction of the hysteresis loop, and the third category represents poor hysteretic behavior.

For the development of the equations for SR_A and SR_V for this code section, as assumption has been made regarding the type of behavior that the existing structure will experience. It has been assumed that the existing buildings will experience poor hysteretic behavior, and the corresponding k factor has been incorporated into the SR_A and SR_V equations.

C-509.3.3.5

No commentary

C-509.3.4 Response limits.

No commentary

C- 509.4 DISPLACEMENT COEFFICIENT ANALYSIS PROCEDURE

C-509.4.1 General

No commentary is provided for this section.

C-509.4.2 Target Displacement (δ_t)

Target displacement is an estimate of the likely displacement of the structure under the design earthquake. It uses data obtained from statistical studies on bilinear and trilinear, non-strength degrading single-degree-of-freedom (SDOF) systems with viscous damping equal to 5% of the critical. The target displacement of the multi-degree-of-freedom

(MDOF) system is computed by modifying the SDOF displacement by using a number of modification factors.

Coefficient C_0 accounts for the difference between the roof displacement of a MDOF building and the displacement of the equivalent SDOF system. Using only the first mode shape and elastic behavior, coefficient C_0 is equal to the first mode participation factor at the roof level. The actual shape vector may take a different form, especially since it is intended to simulate the time-varying deflection profile of the building responding inelastically to ground motion. Based on past studies, the use of a shape vector corresponding to the deflected shape at the target displacement level is more appropriate. This shape will likely be different from the elastic first-mode shape. The tabulated values given in the code (which are based on the number of stories) are based on straight-line vector with equal masses at each floor level. These may be very approximate especially if there is a great variation in the floor masses over the height of the building.

Coefficient C_1 accounts for the difference between peak displacement amplitude for nonlinear response as compared with the linear response in structures with relatively stable and full hysteresis loops. The values of the coefficient are based on analytical and experimental investigations of seismic response of yielding structures. The quantity R in the code represents the ratio of the required elastic strength to the yielding strength of the structure. Some recent studies suggest that maximum elastic and inelastic displacement amplitudes may differ considerably if either the strength ratio R is large or if the building is located in the near-field of a causative fault. If the value of R exceeds five, it is recommended that a displacement larger than the elastic displacement be used as the basis for calculating the target displacement.

Coefficient C_2 represents the effect of the hysteresis shape on the maximum displacement response. If the hysteresis loops exhibit significant pinching or stiffness degradation, the energy absorption and dissipation capacities decrease resulting in larger displacements. This effect is known to be important for short-period, low-strength structures with very pinched hysteresis loops. Since pinching is a manifestation of the structural damage, the smaller the degree of nonlinear response, smaller the degree of pinching. Thus, the values of coefficient C_2 are smaller for systems with periods higher than T_0 and smaller levels of damage.

C-509.4.3 Lateral Load Patterns

The distribution of lateral inertia forces varies continuously during earthquake response. The extremes of the distribution depend upon various factors like severity of earthquake shaking (or degree of nonlinear response), frequency characteristics of the building and the ground motion, etc. The distribution of inertia forces determines relative magnitudes of shears, moments, deformations, etc. The loading profile that is critical for one design quantity may be different from the same that is critical for another design quantity. To account for these issues, code requires that at least two different lateral load patterns be considered. It is assumed that with the two load patterns recommended in the code, the range of design actions occurring during actual seismic response will be approximately bound.

The uniform load pattern is recommended because it emphasizes demands in lower stories over demand in upper stories and magnifies the relative importance of story shear forces compared with overturning moments. The other load patterns, which are based on C_{vx} or modal patterns are recommended so as to give credit to at least the elastic higher mode effects.

C-509.4.4 Period Determination

The fundamental period of the structure changes as the structure deforms into the inelastic range. The elastic response spectra provide only an approximation of response once the structure has entered the nonlinear range, regardless of the reference period used initially. Thus, in order to simplify the analysis process, a reference period corresponding to 60% of the yield strength is recommended. Determination of this period requires that the structure first be loaded laterally to large deformation levels and the overall load-deformation relation be then graphically examined.

It is important to note that it is not appropriate to use empirical code equations for the determination of period because these provide low estimates of fundamental period that result in higher spectral design forces. The code equations are appropriate for the linear analysis procedures because higher spectral forces would result in higher force and deformation demands. On the contrary, it is more conservative to use a high estimate of fundamental period for the nonlinear procedure as that will result in higher target displacement.

C-509.4.5 General Execution Procedure for the Displacement Coefficient Analysis Procedure.

The procedure for pushover analysis presented in the following considers the effect of higher modes. For details, the reader is referred to the paper "Adaptive Spectra-Based Pushover Procedure for Seismic Evaluation of Structures" by Gupta and Kunnath (Earthquake Spectra, May 2000). The main differences between the procedure presented in §809.4 and the procedure recommended in the following section are two-fold. First, the recommended procedure uses a site-specific spectrum to define the loading characteristics, and second, the applied load pattern is changed continuously depending on the instantaneous dynamic properties of the system. In the following procedure, the spectral estimates become the basis for determining the incremental lateral forces to be applied in the pushover analysis itself. Also, the load pattern in the recommended method can consider as many modes as deemed important during the course of the analysis. The basic steps involved in executing the pushover analysis using the recommended procedure are as follows.

1. Create a mathematical model of the structure.

2. Specify the nonlinear force-deformation relations for various elements in the structure. In the simplest form, this entails specifying the initial stiffness, the yield moment and the post-yield stiffness of the element. Alternatively, it is possible to define a more detailed force-deformation envelope that includes cracking, yielding and P-delta softening. Available section analysis program such as BIAX (Wallace and Moehle, 1989) can be used to generate the expected section behavior.
3. Compute the damped elastic response spectrum for the site-specific ground motion to be used for evaluation. This is required to obtain the modal spectral accelerations (elastic force demands) at various steps. Suitable damping constants, depending upon the structural material and type, should be used. For example, 5% of critical damping is a reasonable value for RC structures.
4. Perform an eigen-value analysis of the structural model at the current stiffness state (for the first step this will be the initial stiffness) of the structure to compute periods and eigen values of the system. Using the story weights (masses) and the computed eigen values, determine the modal participation factors as given by the following expression:

$$\Gamma_j = \frac{1}{g} \sum_{i=1}^{i=N} W_i \phi_{ij} \tag{1}$$

where:

Γ_j = modal participation factor for j^{th} mode
ϕ_{ij} = mass normalized mode shape value at i^{th} level and for j^{th} mode
W_i = Weight of i^{th} story
g = Acceleration due to gravity
N = Number of Stories
Note that Equation (1) has been normalized such that $\Sigma W \Phi^2 = 1$.

5. Compute the story forces at each story level for each of the n modes to be included in the analysis using the following relationship:

$$F_{ij} = \Gamma_j \, \phi_{ij} \, W_i \, S_a(j) \tag{2}$$

where:
F_{ij} = lateral story force at i^{th} level for j^{th} mode ($1 \le j \le n$)
$S_a(j)$ = spectral acceleration corresponding to j^{th} mode

6. Compute modal base shears (V_j) and combine them using SRSS to compute building base shear (V) as shown below:

$$V_j = \sum_{i=1}^{i=N} F_{ij} \tag{3}$$

$$V = \sqrt{\sum_{j=1}^{N} V_j^2} \tag{4}$$

7. The story forces computed in Step 5 are uniformly scaled using the scaling factor S_n indicated below:

$$\overline{V}_j = S_n V_j \tag{5a}$$

where:

$$S_n = \frac{V_B}{N_S V} \tag{5b}$$

and V_B is the base shear estimate for the entire structure and N_S is the number of uniform steps over which the base shear is to be applied.

NOTE: The process of applying lateral forces to the structural model commences at this step in the first iteration. The lateral force is applied incrementally in small steps to avoid excessive overshooting of element yield forces. Initial increments during the elastic phase of the response can be considerably larger than the final increments in the post-yield phase. In the procedure described here, equal increments are assumed for simplicity. For example, assume that the base shear estimate is 40% of W (W being the seismic weight of the building) and the number of steps in which the total force will be applied is 100. If the building base shear computed in step 6 above is 25% of W, then all the story forces would be scaled by a factor of 0.016 (=0.40/0.25/100). Scaled story forces would then result in a building base shear of 0.004W, which is equal to the base shear to be applied in one increment.

8. Perform a static analysis of the structure using the scaled incremental story forces computed in the previous step corresponding to each mode independently. This means that for modes other than the fundamental mode, the structure will be pushed and pulled simultaneously.
9. Compute element forces, displacements, story drifts, member rotations, etc. by an SRSS combination of the respective modal quantities for this step and add these to the same from the previous step.
10. At the end of every step, compare the accumulated member forces with their respective yield values. If any member has yielded, re-compute the member and global stiffness matrices and return to Step 4.
11. Repeat the process until either the maximum base shear has been reached or the global drift exceeds the specified limit.

It is clear from the above description that the applied load pattern keeps varying continuously based on the instantaneous dynamic characteristics of the structure. The computation of the story forces is, at any step, identical to traditional response spectrum analysis. Whenever one or more elements yield, a new structure is created by changing the stiffness of the yielded element(s) and the response spectral analysis is repeated. Since one ground motion results in one pushover curve, a suite of ground motions will produce a family of curves, which can be used to generate mean response parameters.

C- 509.4.6 Acceptance criteria.

The compressive strains in the components of the structural system caused by the interstory drift associated with a structural system displacement of 150 percent of the target displacement shall be equal or less than that specified in Section 509.1.2.2 Items 1 through 3. The tangent to the load-displacement curve at 150 percent of the target displacement shall not have a negative slope except as permitted by the exception to Section 509.1.2.2 Item 4.

An exception to the requirement that 150 percent of the target displacement be used for acceptance is given for a structure when rational analysis can determine an effective stiffness and a yield strength for the components of the structure at the target displacement and these calculated stiffness and strengths correspond to that predicted by the structural model at the target displacement. A deviation of less than 20 percent between the independently calculated values and the system values should be considered confirmation of the system performance at the target displacement.

C- 510- DETERMINATION OF THE STRESS-STRAIN RELATIONSHIP OF EXISTING UNREINFORCED MASONRY

C- 510.1 Scope.

The unreinforced masonry used in existing infill buildings may have stress-strain characteristics significantly different from those used in design codes for new buildings. Earthquake hazard reduction guidelines uses expected material values for stress-strain relationships. These expected material values also should include apparent yield and peak strength. The analyses of existing buildings anticipates nonlinear response to probable earthquake shaking. A nonlinear analysis of an element such as a confined panel of unreinforced masonry requires a simulation of cyclic compressive loading be used to determine the degradation, if any, of the compressive modulus of the masonry.

C-510.2 General procedure.

This section is applicable for solid masonry units. Grouted masonry, reinforced or unreinforced, is rarely found in infilled panels and its testing is described in Section 505.2.2.Flat jacks have not been successively used in partially grouted or ungrouted cored brick or hollow unit masonry. Use of bearing plates to span cores or cells of masonry units to provide a relatively uniform confinement of the surface of the flat jack generally will require a space greater than the height of the existing mortar joint.

The location of the test should be designated by the engineer. The available overburden or vertical confinement above the flat jack should be determined by the engineer.

C- 510.3 Preparation for the test.

The smoothness of the cut surface and a consistent width of the cut made for insertion of the flat jack and the steel shims is critical. Use of partial length shims in a cut without parallel edges will result in misleading data. Use of a masonry saw resting on a guide attached to the masonry wall is the most successful method of cutting the slot.

The restriction on the parallelism of the cuts given in this section is for the two cuts made for isolating the prism from the wall.

C- 510.4 Required equipment.

Flat jacks are commonly manufactured to order for the project but useable jacks may be in the inventory of testing agencies. The life of a flat jack is relatively short. Failure (leakage) is common due to fracture of the weld joining the upper and lower surfaces of the flat jack. The life of the flat jack is extended by using multiple steel shims of varying thickness to completely fill the cut made for the flat jack.

C-510.5 Data acquisition equipment.

A critical step in obtaining useful stress-strain data is the mounting of strain gauges at the prescribed distance from the face of the wall (one-sixteenth inch). The five unit high prism will bulge from Poisson effect as the strain approaches peak values. Bulging of the prism may cause a relative rotation of the extensions of the gauge points on the wall. This relative rotation could alter the measured value from the strain in the prism.

C- 510.6 Loading and recording data.

The flow of hydraulic fluid to the flat jack must be controlled as in a cyclic test using displacement control. A hand operated pump is adequate and recommended. Observation of the pressure gauge is used for determination of the specified incremental loading. Flow of hydraulic fluid to the flat jack must be stopped for recording of strain data.

It is very important to obtain incremental unloading and reloading stress-strain data. Recording permanent set at the end of each loading cycle is also very important. Permanent compressive set will cause the hysteretic behavior of the compressive strut within the infill panel to become pinched and degrade the secant stiffness of the infilled panel.

C- 510.7 Quality control.

No commentary

C- 510.8 Interpretation of the data.

The total quantity of stress-strain data obtained shall be averaged for calculation of expected peak compressive stress and strain. An averaged shape of the envelope of the stress-strain relationship should be prepared. A nonlinear push-over analysis of the infilled frame requires that this envelope be used for stiffness degradation of the infill element.

Experimental testing by flat jacks has shown that use of three strain measurement devices on the face of the prism gives data that should not have equal weighting. The loading effects of the flat jack extends beyond the length of the flat jack. This effect is minimized by using strain gauges at one-third length points on the prism.