A PROGRESS REPORT ON
ATC 55: EVALUATION AND IMPROVEMENT OF INELASTIC SEISMIC ANALYSIS PROCEDURES (FALL 2002)

CRAIG D. COMARTIN[1]

INTRODUCTION AND BACKGROUND

The objectives of the ATC 55 project are the development of practical recommendations for improved prediction of inelastic structural response of buildings to earthquakes (i.e., guidance for improved application of simplified inelastic analysis procedures) and the identification of important issues for future research. Specific anticipated outcomes are:

1. Improved understanding of the inherent assumptions and theoretical underpinnings of existing and proposed new simplified analysis procedures.
2. Recognition of the applicability, limitations, and reliability of various procedures.
3. Guidelines for practicing engineers to apply the procedures to new and existing buildings.
4. Direction for researchers on issues for future improvements of simplified inelastic analysis procedures.

The results of the project will culminate in a project document to be published by FEMA. This document will provide a comprehensive discussion of simplified inelastic seismic analysis of new and existing buildings. It will contain guidelines for applications of selected procedures including their individual strengths, weaknesses and limitations. The document will also contain illustrative examples and expert commentary on key issues. The document will serve to update and supplement existing publications including *FEMA 273/274, ATC 40,* and the *NEHRP Recommended Provisions*

The first phase of the project comprised an assessment of pertinent aspects of the state of research and practice. Information on the project and the results of the first phase may be accessed at the ATC web site (www.atcouncil.org). As of October 2002, the second phase of the project is nearing completion. This phase has focused upon the detailed evaluation of current procedures and the development of recommended improvements. The results of the evaluation process are documented by Miranda (2002ab). This paper summarizes the current status of the proposed improvements. These are being developed currently by the project team.

Contemplated improvements include better estimates of inelastic displacements when using nonlinear static procedures (NSP's). There are currently two alternatives. *FEMA 356* documents the Displacement Coefficient Method (DCM). The basis of this

[1] *Comartin-Reis, Stockton, California*
comartin@comartin-reis.com

approach is the statistical analyses of the results of time histories of SDOF oscillators used to generate inelastic spectra or *R*-μ-*T* relationships. The results are used to formulate coefficients used to modify the response of a linear system. This basic approach is termed **displacement modification**. The other alternative is documented in *ATC 40* as the Capacity Spectrum Method (CSM). This approach relies on **equivalent linearization** of the inelastic system utilizing both a period shift (decrease in stiffness) and equivalent viscous damping to represent hysteretic energy loss. These parameters are specific to each system. They are also a function of ductility and current methods require iteration for solution. The development of improved procedures for both are outlined in the following sections of this paper.

The current NSP's of both *FEMA 356* and *ATC 40* rely primarily on single-degree-of-freedom analysis of response. Both documents touch upon variations in load vectors or other attempts to recognize the effects of higher modes of vibration. The results from Phase I indicate that there may be potential improvements. First, several studies suggest that modification of the load vector during the pushover analysis to reflect changes in the vertical distribution of forces resulting from inelastic behavior can improve NSP results compared with actual MDOF analyses. Secondly, other studies appear to show that combining the results of several pushovers representative of different mode shapes for the same structure can lead to improved comparisons with actual MDOF analyses. These potential improvements are outlined in a subsequent section on **multi-degree-of-freedom effects**.

For a number of reasons, short period buildings may not respond to seismic shaking as adversely as might be predicted analytically. Traditional design and evaluation procedures, including *FEMA 356*, recognize this with various provisions. These provisions are not based directly upon empirical or theoretical justifications. In order to at least begin to address this shortcoming, the ATC 55 project scope has been recently expanded with an effort to document and discuss **short period effects** within an improve technical context. The last section of this paper provides an outline of the issues currently being investigated.

DISPLACEMENT MODIFICATION

The ATC 55 project team is contemplating several recommended improvements (Miranda 2002c) to the displacement modification procedure in *FEMA 356* (BSSC 2000). These relate to the basic coefficient method equation for the target displacement, δ_t in estimating the maximum inelastic global deformation demands on buildings for earthquake ground motions

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

where the coefficients are currently defined as follows:

C_o = modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system.

C_1 = modification factor to relate the expected maximum inelastic displacements to displacements calculated for linear elastic response.

= 1.0 for $T_e \geq T_s$

= $[1.0 + (R-1)T_s/T_e]/R$ for $T < T_s$

C_1 computed by the above equations can by capped the limiting equations

$C_1 = 1.5$ for $T_e < 0.1$s.

$C_1 = 1.0$ for $T_e \geq T_s$.

Linear interpolation is allowed for the intermediate values $0.1 < T_e \leq T_s$.

R = Ratio of elastic strength demand to calculated yield strength.

T_e = Effective fundamental period of the building.

T_s = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity spectrum of the spectrum.

C_2 = Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on the maximum displacement response. Values of C_2 for different framing systems and Structural Performance Levels (i.e. immediate occupancy, life safety and collapse prevention) are obtained from Table 3.3 of the FEMA-356. Alternatively, C_2 can take the value of one.

C_2 values from FEMA-356 (BSSC, 2000)

Structural performance level	$T \leq 0.1$second		$T > T_s$	
	Framing Type 1[1]	Framing Type 2[2]	Framing Type 1[1]	Framing Type 2[2]
Immediate occupancy	1.0	1.0	1.0	1.0
Life safety	1.3	1.0	1.1	1.0
Collapse prevention	1.5	1.0	1.2	1.0

[1] Structures in which more than 30% of the shear at any level by combination of the following components, elements or frames: ordinary moment resisting frame, concentrically moment braced frame, frames with partially restrained connections, tension only braces, unreinforced masonry walls, shear-critical, piers and spandrels of reinforced concrete and masonry.

[2] All frames not assigned to Frame Type 1.

[3] Linear interpolation shall be used for intermediate values of T.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 is set equal to 1. For buildings with negative post-yield stiffness, values of C_3 is calculated using the following expression:

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e}$$

Based on the analyses of the current procedures (Miranda 2002a) two alternatives for improvement of the factor C_1 are being considered:

ALTERNATIVE 1:

$$C_1 = 1 + \left[\frac{1}{a \cdot (T_e / T_g)^b} - \frac{1}{c} \right] \cdot (R-1)$$

Soil profile	***a***	***b***	***c***	***T_g (s)***
B	42	1.60	45	0.75
C	48	1.80	50	0.85
D	57	1.85	60	1.05

ALTERNATIVE 2:

$$C_1 = 1 + \left[\frac{1}{a \cdot (T_e / T_g)^b} \right] \cdot (R-1)$$

Soil profile	***a***	***b***	***T_g (s)***
B	151	1.60	1.60
C	199	1.83	1.75
D	203	1.91	1.85

where T_g = a site dependent period.

These are both compared to the current definition in Figure 1. Figure 2 illustrates the substantial improvement in error reduction with either alternative

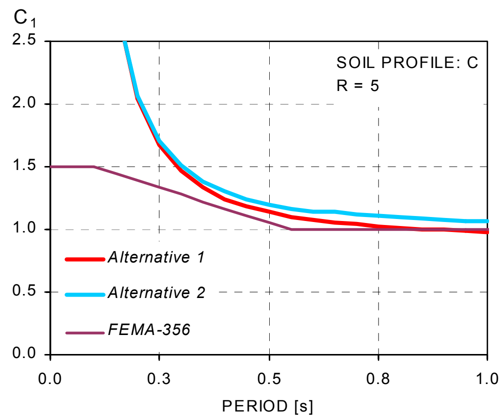

Figure 1: Comparison of current and potential C_1 coefficients (from Miranda 2002c)

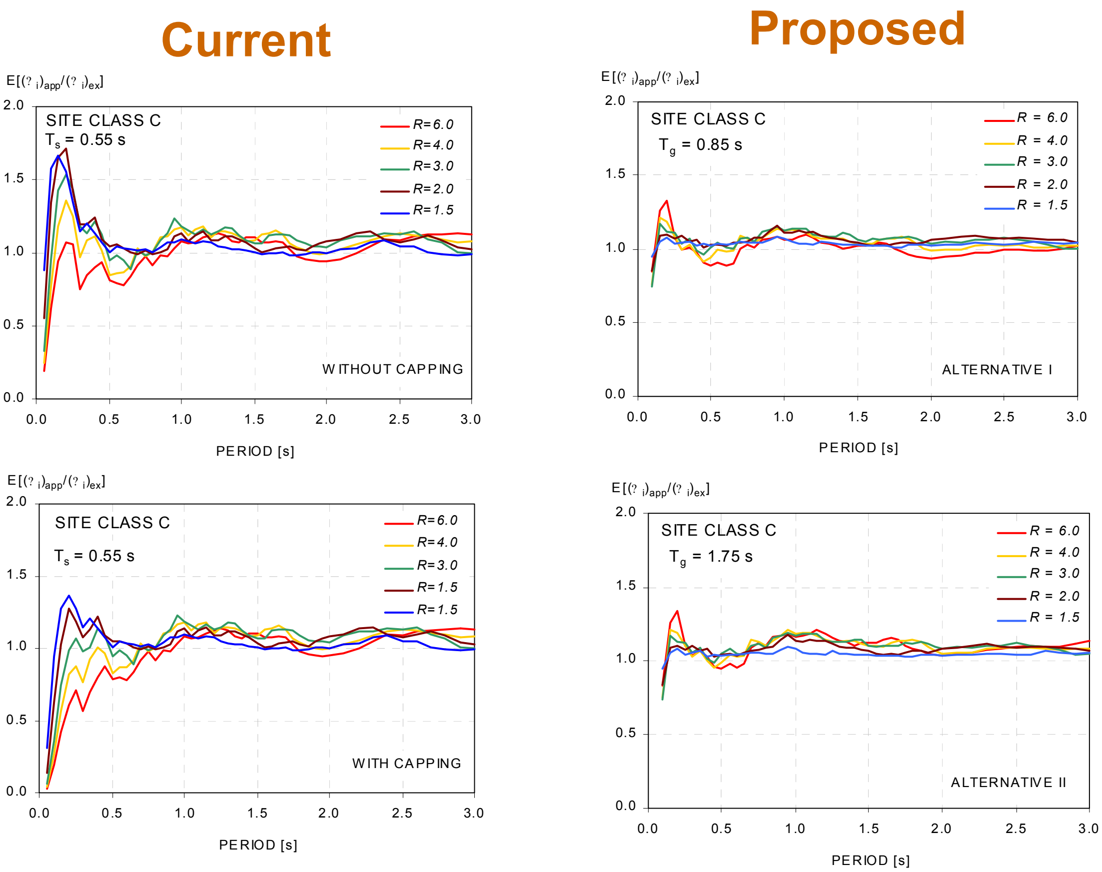

Figure 2: Comparison of mean errors for C_1 coefficients for site class C (from Miranda 2002c)

Miranda (2002a) points out that the current definitions C_2 and C_3 are not clearly independent of one another. C_2 is intended to represent changes in hysteretic behavior due to pinching, stiffness degradation, and strength degradation. However, strength and stiffness degradation due to P-Δ effects are supposedly addressed by C_3 as well. The proposed improvements include a clearer separation of these coefficients outlined as follows:

C_2 = Modification factor to represent CYCLIC DEGRADATION (both stiffness and strength degradation).

ALTERNATIVE 1:

$C_2 > 1$ for $Te < 0.5$s

$C_2 = 1$ for $Te \geq 0.5$s

ALTERNATIVE 2:

$C_2 = 1$

Figure 3 illustrates that for stiffness degrading (SD) and strength-and-stiffness degrading (SDD) behavior C_2 is actually less than 1.0 except for low strength short period oscillators.

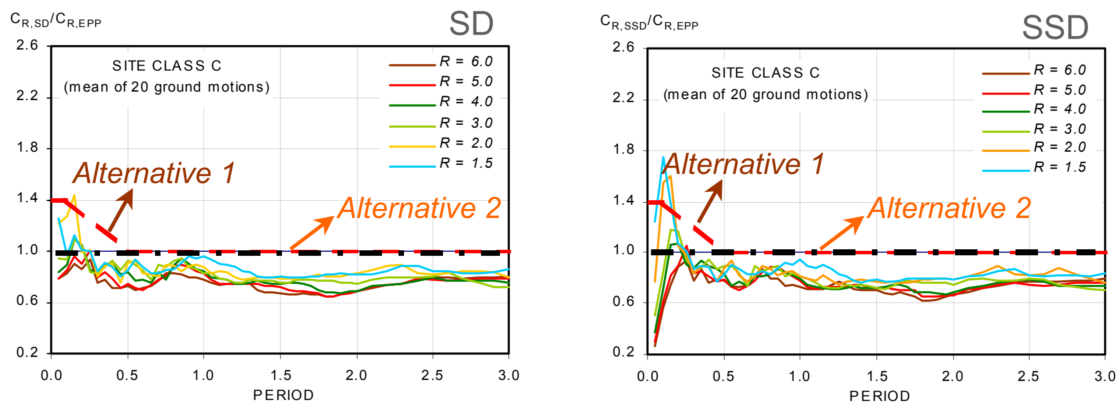

Figure 3: Comparison of proposed C_2 coefficients with the results of RHA (from Miranda 2002c)

C_3 = Modification factor to represent the effect of STRENGTH DEGRADATION WITHIN A CURRENT HALF-CYCLE, when there is a negative stiffness in the pushover curve. For buildings with positive yield stiffness, C_3 is set equal to 1.

The negative stiffness can come from geometric nonlinearities (i.e., P-Δ effects), material nonlinearities (strength degradation, brittle failures, etc) or combination of these phenomena. The analytical definition of this coefficient is being studied based on a number of parameters that control the point at which instability (collapse) occurs. The general shape of the relationship to strength is shown in Figure 4. This figure suggests that an alternative to the C_3 coefficient might be to impose some limitations on directly on the strength of buildings with negative post-elastic stiffness to avoid collapse. This is also under consideration.

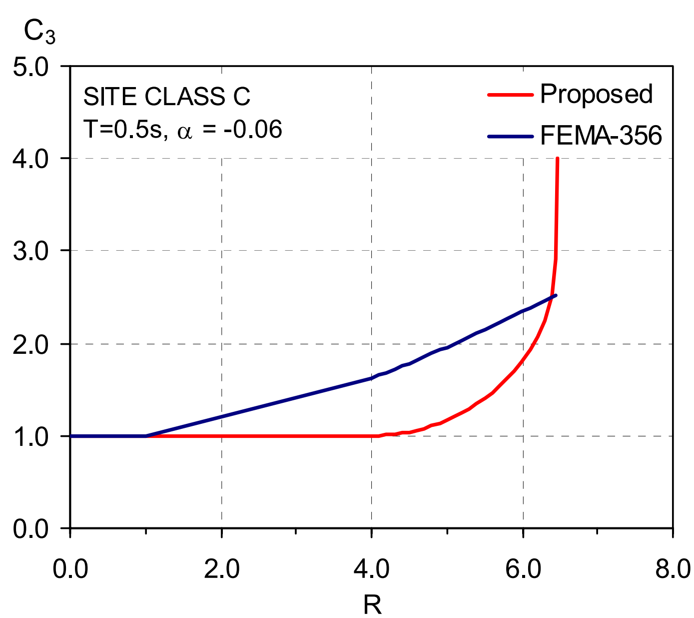

Figure 4: Comparison of general shape of current and proposed C_3 coefficients (from Miranda 2002c)

EQUIVALENT LINEARIZATION

The capacity spectrum method documented in *ATC 40* (ATC 1997) is a form of equivalent linearization based on two fundamental assumptions. The period of the equivalent linear system is assumed to the secant period and the equivalent damping is related to the area under the capacity curve associated with the inelastic displacement demand. The focus of the ATC 55 effort (Iwan 2002) has been to develop better procedures to estimate equivalent period and equivalent damping. This is an extension of previous work (Iwan 1978 and 1980) in which both parameters are expressed as functions of ductility. These relationships are based on an optimization process whereby the error between the displacement predicted using the an equivalent linear oscillator and using nonlinear response history analysis is minimized. Conventionally, the measurement of error has been the mean of the absolute difference between the displacements. Although this seems logical, it might not lead to particularly good results from an engineering standpoint. This is illustrated in Figure 5 from. It is possible to select linear parameters for which the mean error is zero as for the broad, flat distribution. However, the narrower curve might represent equivalent linear parameters that provide better results from an engineering standpoint, since the chance of errors outside say a –20% to +10% range are much lower. This is owing to the smaller standard deviation in spite of the –5% mean error.

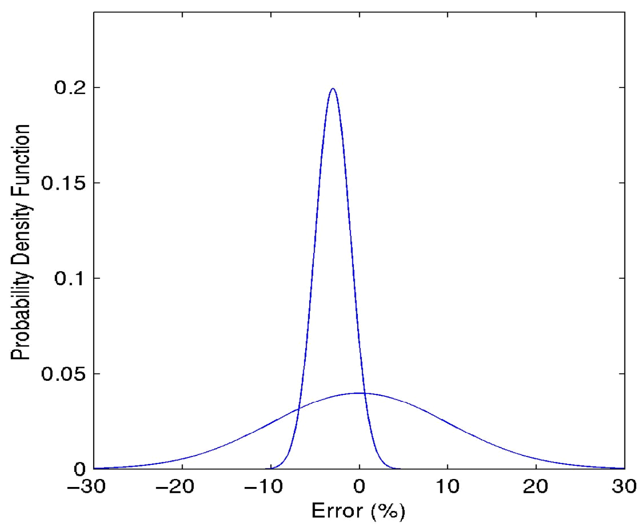

Figure 5. Illustration of probability density function of displacement error for a Gaussian distribution-from (Iwan 2002).

This general strategy has been applied to a series of elasto-plastic, stiffness degrading, and strength-and-stiffness-degrading hysteretic models generate optimal equivalent linear parameters for a range on periods and ductilities as illustrated in Figure 6.

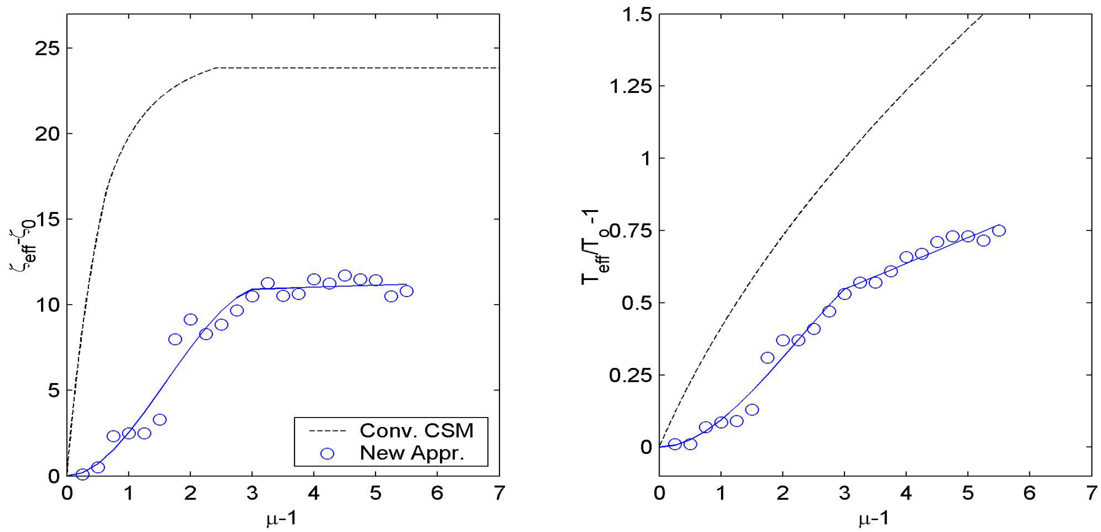

Figure 6: New optimal effective (equivalent) linear parameters for elastoplastic system.T_0=0.1-2.0 (from Iwan 2002).

Using the results for discrete values of ductility, a curve fitting process has leads to analytical expressions relating effective period, T_{eff}, and effective damping, ξ_{eff}, to ductility, μ, similar to the following:

For μ<4.0:

$$T_{eff}/T_o - 1 = 0.111(\mu-1)^2 - 0.0167(\mu-1)^3$$
$$\zeta_{eff} - \zeta_o = 3.19(\mu-1)^2 - 0.660(\mu-1)^3$$

For μ≥4.0:

$$T_{eff}/T_o - 1 = 0.279 + 0.0892(\mu-1)$$
$$\zeta_{eff} - \zeta_o = 10.6 + 0.116(\mu-1)$$

In practical applications, the parameter of interest is most often the maximum inelastic displacement which is directly related to ductility. Consequently, the application of these expressions generally require iteration, as with the previous capacity spectrum method. In contrast to the previous procedure however, the use of the optimal effective period and damping directly produces a point on an acceleration and displacement response diagram (ADRS) that does not lie on the capacity spectrum for the structure (see Figure 7). Although the interection of T_{eff} with the ADRS demand reduced by ξ_{eff} identifies the proper maximum displacement, $D_{max,}$ the corresponding maximum acceleration, A_{max}, must lie on the capacity spectrum. This may be easily corrected graphically by multiplying the value of the acceleration at every displacement on the reduced ADRS by the ratio of the corresponding secant period, T_{sec}, of the capacity spectrum at that displacement to the effective period, T_{eff}, for the same displacement. This results in what has been termed a Modified ADRS (MADRS) that is a function of ductility and the specific capacity spectrum. Thus a family of curves maybe generated for a given structure as shown in Figure 8. The intersection of the radial effective period lines and the MADRS curves corresponding to the same ductilities trace the locus of potential performance points. The actual performance point for the structure is then the intersection of this locus and the capacity spectrum. Characterized in this manner the application improved is analogous to the previous capacity spectrum method.

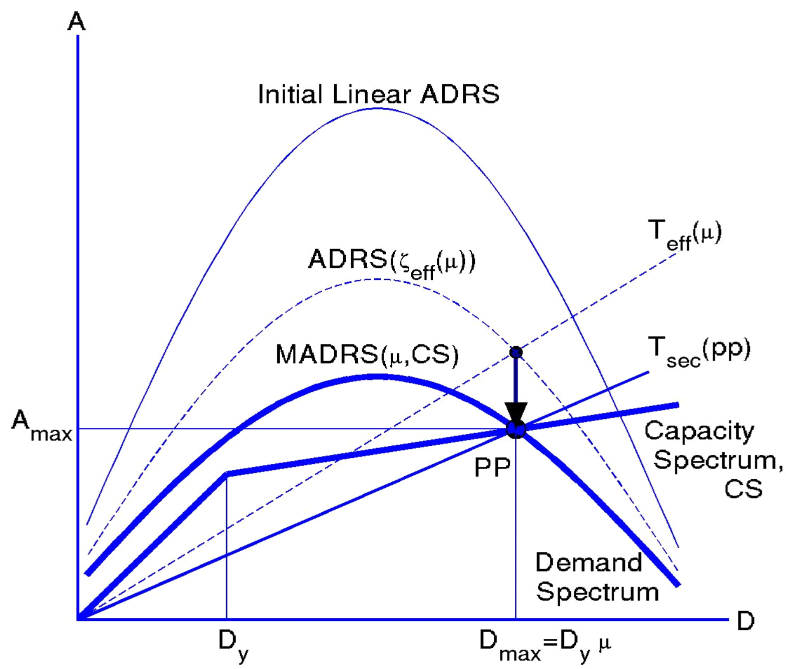

Figure 7: Description of the Modified ADRS (MADRS) and its use (from Iwan 2002).

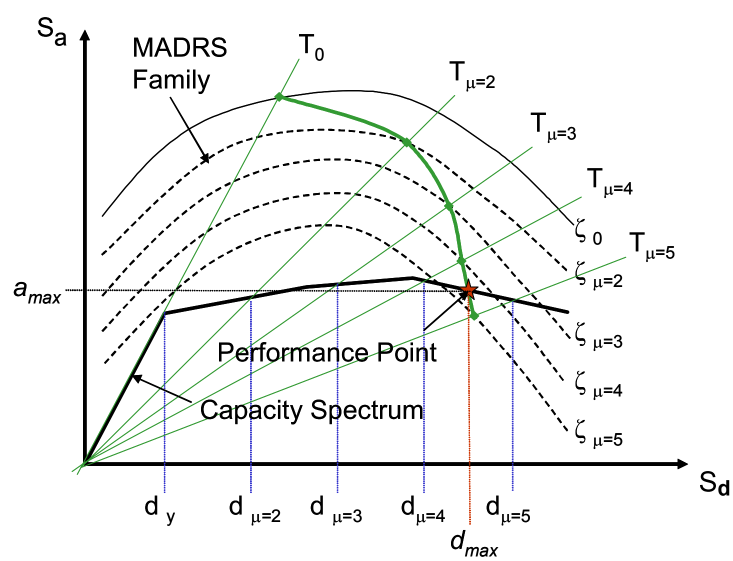

Figure 8: Illustration of a graphical procedure for finding the Performance Point using a family of MADRS (from Iwan 2002).

Figure 9 provides a comparison between the previous capacity spectrum method approach of ATC 40 and the proposed improved MADRS procedures for the UBC design spectrum.

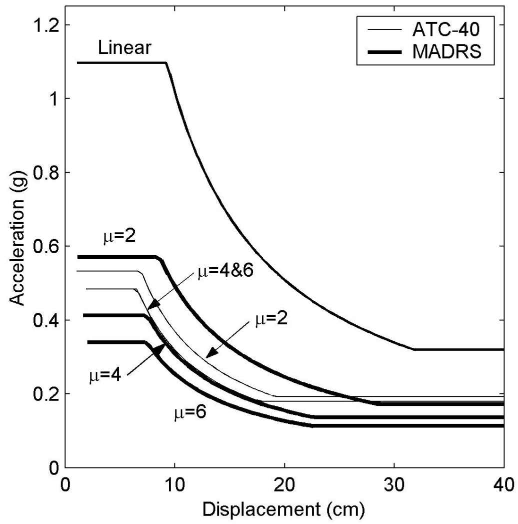

Figure 9: UBC based reduced ADRS from conventional ATC-40 approach and MADRS from new optimal parameters for an elastoplastic system (from Iwan 2002).

For low levels of ductility it is evident that the MADRS procedure will predict relatively higher displacements. However, for higher ductility demands the improved procedures will predict significantly lower displacements than the ATC 40 approach. This effect is most evident for systems with very short initial periods (high initial stiffness) or long initial periods (low stiffness). These differences can be important in evaluating the performance of older buildings and may partially address the question of why the conventional CSM approach appears to be overly conservative for some short period, low strength structures based on actual earthquake performance. This issue needs to be examined further.

MULTI-DEGREE-OF-FREEDOM EFFECTS

In order to compare and illustrate techniques for improving the results of nonlinear static procedures related to the effects of higher modes, five example buildings have been analyzed (Aschheim 2002). The basic outline of this effort is as follows:

Objective — Compare estimates made using simplified inelastic procedures with results obtained by nonlinear dynamic analysis

Example Buildings

3-Story Steel Frame (SAC LA Pre-Northridge M1 Model)

3-Story Weak Story Frame (lowest story at 50% of strength)

8-Story Shear Wall (Escondido Village)

9-Story Steel Frame (SAC LA Pre-Northridge M1 Model)

Ground Motions

11 Site Class C Motions, 8-20 km, 5 events

4 Near Field Motions: Erzincan, Northridge (Rinaldi Receiving Station & Sylmar County Hospital), and Landers

Drift Levels

Ordinary Motions (scaled)

0.5, 2, 4% for frames

0.2, 1, 2% for wall

Near-Field (unscaled)

1.8 to 5.0% for 3-story frames, 1.7-2.1% for 9-story frames

0.6 – 2.1% for wall

Load Vectors/Methods Illustated

First Mode

Inverted Triangular

Rectangular (Uniform)

Code

Adaptive

SRSS

Multimode Pushover (MPA)

Response Quantities (Peak values generally occur at different instants in time)

Floor and roof displacements

Interstory Drifts

Story Shears

Overturning Moment

Errors

Mean over all floors

Maximum over all floors

Major observations from MDOF examples are summarized as follows:

Displacements

Displacements are estimated well by approximate methods, except:

Displacement response is not always predominantly in a first mode.

Weak story mechanisms can occur for some motions and not others. Pushover analyses show weak story mechanisms.

The load vectors result in similar displacement estimates.

The rectangular, code, and SRSS vectors are a little worse than the others.

The adaptive does not result in a substantial difference.

Displacements— 3-story frames

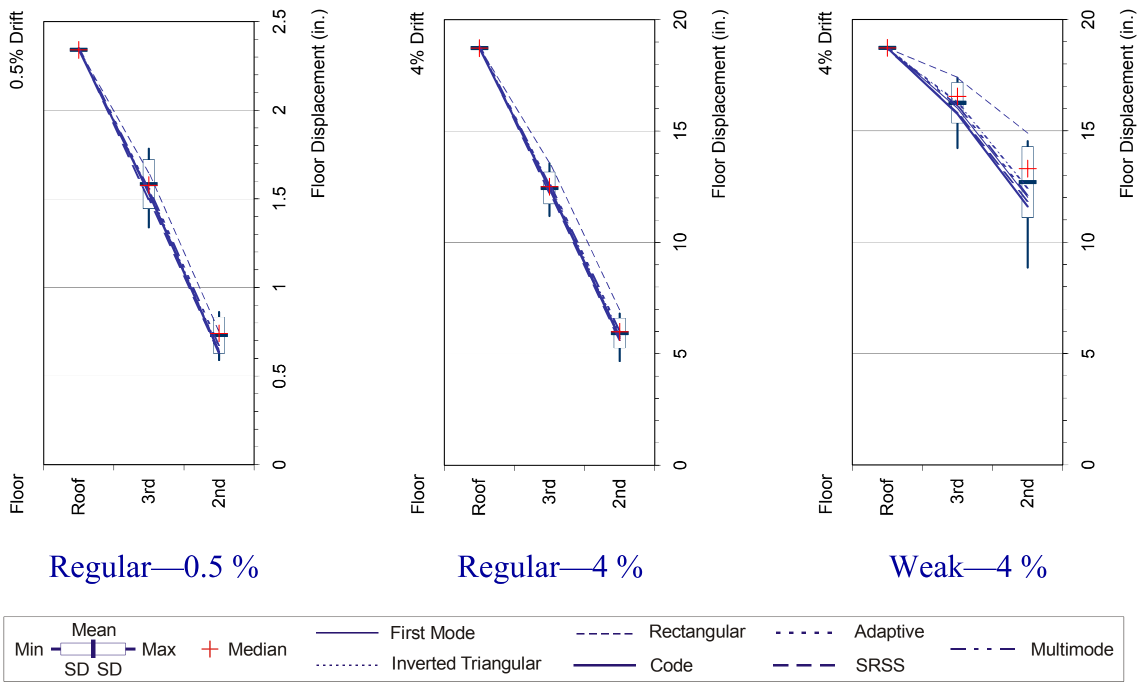

Figure 10: Example results from MDOF examples for displacements (from Aschheim 2002).

Interstory drifts

8-story wall:

Interstory drifts were dominated by the first mode and were estimated well by quasi-first mode vectors. (Interstory shears are estimated poorly by these vectors)

Weak-story frames:

Interstory drifts at the weak story were estimated well by all load vectors. Elsewhere could be severely underestimated.

Regular frames:

Interstory drifts were underestimated by quasi-first mode load vectors.

While much better, even the modified MPA could significantly underestimate interstory drifts for the 9-story frames.

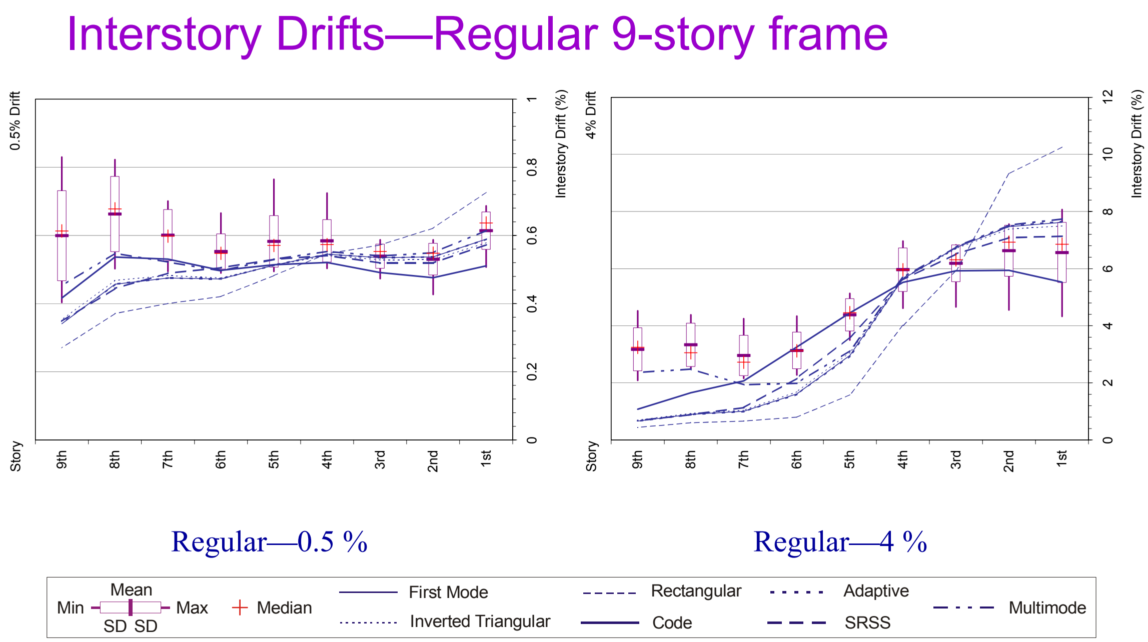

Figure 11: Example results from MDOF examples for interstory drifts (from Aschheim 2002).

Story shears

Story shears generally were underestimated by quasi-first mode load vectors (except at the weak story of the weak-story frames).

A modified MPA method overestimated story shears for the 3-story frames, and could underestimate or overestimate story shears for the 8 and 9-story buildings. (Improvements might involve more modes, with each reduced as nonlinearity increases.)

The Code ELF procedure significantly underestimates shears at large drifts.

A revised F_t approach would require as much as 75% of the base shear to be applied at the top.

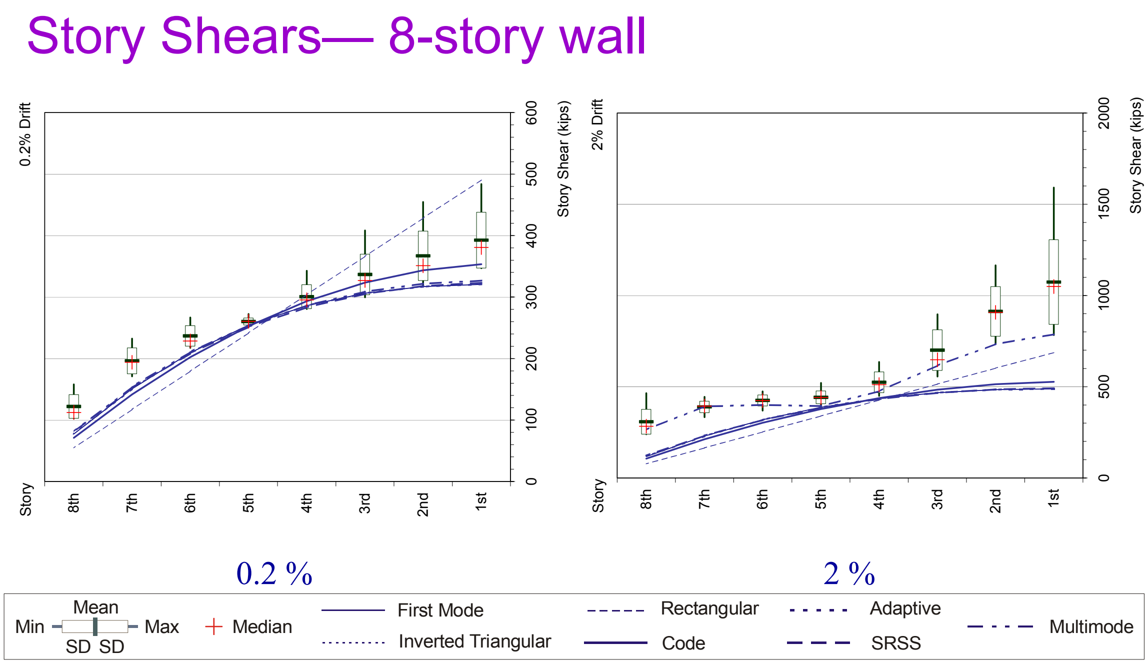

Figure 12: Example results from MDOF examples for story shears (from Aschheim 2002).

Overturning moment

Underestimated by quasi-first mode techniques

MPA is can be accurate, but can also significantly underestimate or overestimate overturning moments.

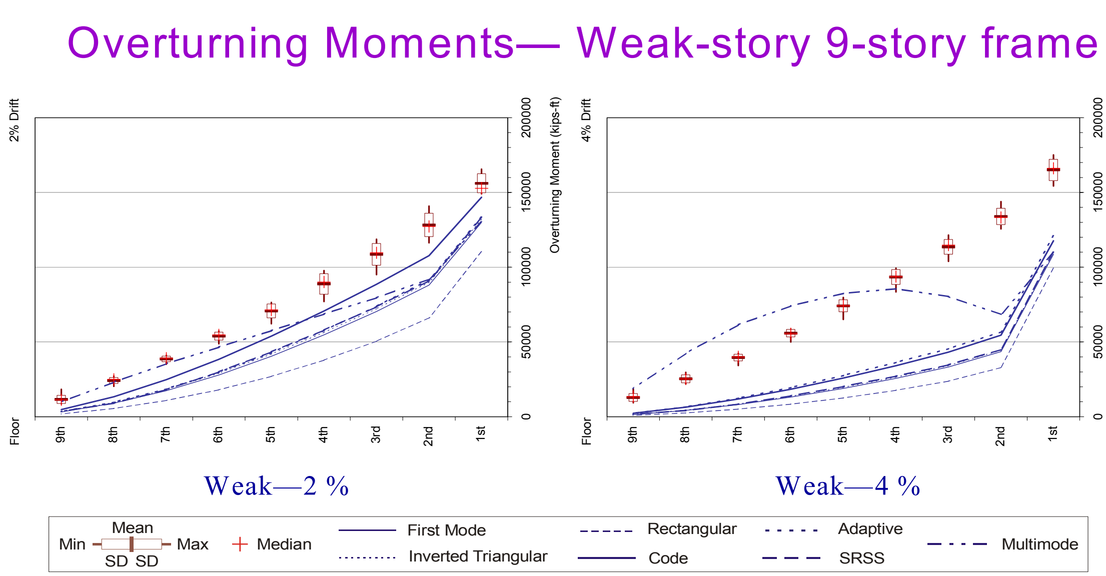

Figure 13: Example results from MDOF examples for overturning (from Aschheim 2002).

Key observations and implications

Displacement response usually is dominated by a first mode. Displacements are estimated well.

Pushover analysis shows weak story mechanisms that do not always occur dynamically.

ESDOF estimates:

For positive post-yield stiffness: are slightly conservative, are applicable to ordinary and near-field motions.

For negative post-yield stiffness, ESDOF estimates can be much too large.

Peak displacements generally estimated well by all load vectors. Complex or multiple load vectors are not needed.

Errors for interstory drifts, story shears, and overturning moments can be substantial. Complex or multiple load vectors still do not give reliable estimates.

SHORT PERIOD EFFECTS

FEMA 356 currently contains limitations (caps) on the maximum value of the coefficient C_1, the ratio of the maximum inelastic displacement of a single degree of freedom elasto-plastic oscillator to the maximum response of the fully elastic oscillator. The authors of *FEMA 356* apparently included the capping limitations for two related reasons. First, there is a belief in the practicing engineering community that short stiff buildings simply do not respond to seismic shaking as adversely as might be predicted analytically. Secondly, authors felt that the required use of the empirical equation without out relief in the short period range would motivate practitioners to revert to the more traditional, and apparently less conservative, linear procedures. Although there may be technical justification for limitations on the maximum value of C_1 particularly for short period structures, the current limitations are not adequately founded on theoretical principles or empirical data. Capping leads to prediction of maximum inelastic displacements that are less than the current empirical relationship by a margin that varies widely depending on period, strength, and site conditions. For periods of interest for most buildings (>0.3 sec. or so), the margin ranges from relatively small (<20%) for firm (Class B) sites to rather large(>200%) for soft (Class E) sites (see Figure 14).

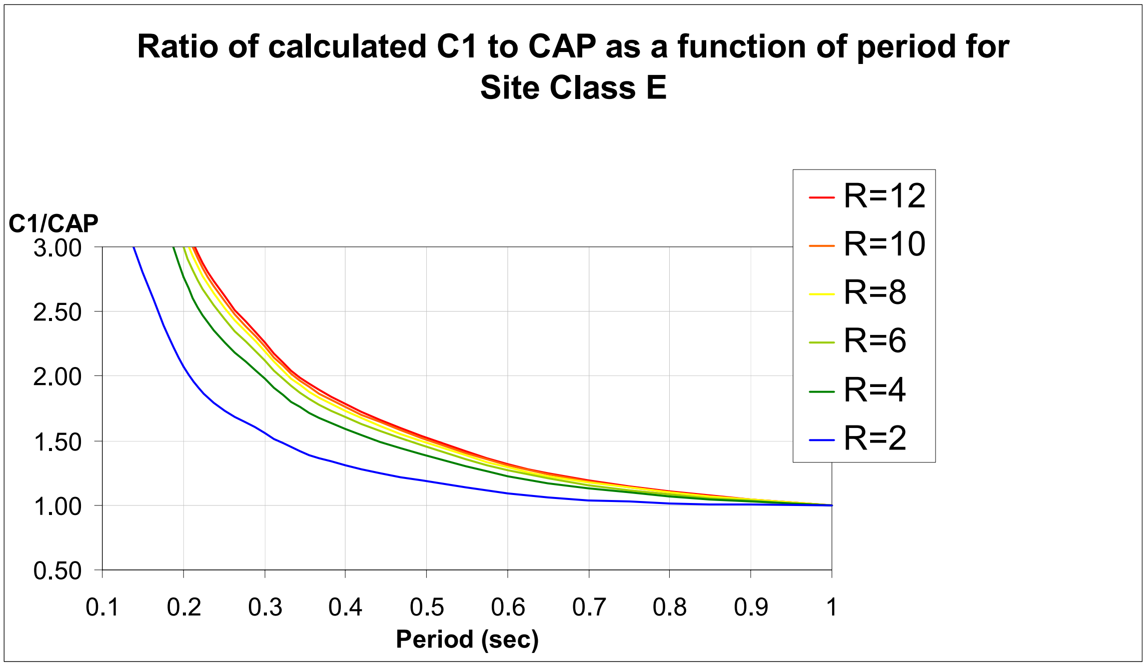

Figure 14: Example of error introduced by capping

There are several interrelated reasons why inelastic displacements for apparently short period buildings might be less than predicted by nonlinear analyses of idealized SDOF systems.

1. Practicing engineers tend to neglect the ascending branch of design spectra when considering first mode response and use the acceleration plateau in this region, assuming that period lengthening resulting from nonlinear behavior will shift the structure to the spectral plateau, during response.
2. Short, stiff buildings generally are more sensitive to interaction between soil material strength and stiffness with that of the structure and its foundations than are longer period structures.
3. Radiation and material damping in supporting soils cause the motion imparted to structures to differ from that of the free field.
4. Full and partial basements, and foundation depth more generally, can modify the motion that a structure feels compared to that in the free field.
5. Building foundations can act as filters effectively cutting off motions at a characteristic period related to the plan dimension of the foundation relative to the shear wave velocity of the supporting soils.
6. Conventional structural analysis procedures lump building masses at floor and roof levels.

The ATC 55 Project is investigating these in a effort to provide guidance and practical procedures for including short period effects more rationally in inelastic analyses. This effort is funded by the Pacific Earthquake Engineering Research Center and is being done under the direction of Jonathan Stewart at the University of California, Los Angeles.

REFERENCES

Aschheim, M., T. Tjhin, and M. Inel, ATC-55 MDOF Studies (ATC 55 draft working paper), University of Illinois-Urbana, 2002

ATC, 1996, *The Seismic Evaluation and Retrofit of Concrete Buildings, Volume 1 and 2*, ATC-40 Report, Applied Technology Council, Redwood City, California.

BSSC (2000), *A prestandard and commentary for the seismic rehabilitation of buildings*, prepared by the Building Seismic Safety Council; published by the Federal Emergency Management Agency, FEMA 356 Report, Washington, DC.

Comartin, C.D., ATC 55: Evaluation and Improvement of Inelastic Seismic Analysis Procedures, US-Japan Workshop, Seattle, August, 2001.

Iwan, W. D. (1980). Estimating inelastic response spectra from elastic spectra. *Intl. J. Earthq. Engng. and Struc. Dyn.*, **8**, 375-388.

Iwan, W. D. and N. C. Gates (1978), The effective period and damping of a class of hysteretic structures, *Intl. J. Earthq. Engng and Struc. Dyn.*, **7**, 199-211.

Iwan, W. D., and A. C. Guyader, An improved equivalent linearization procedure for the capacity spectrum method (ATC 55 draft working paper), California Institute of Technology, Pasadena, California, USA, 2002.

Miranda, E., Evaluation of methods to estimate target displacement in PBEE, 4th US-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforced Concrete Buildings, Toba, Japan, October, 2002a.

Miranda, E., S. Akkar and J. Ruiz-García, Summary of evaluation of current nonlinear static procedures (ATC 55 draft working paper), Stanford University, 2002b.

Miranda, E., S. Akkar and J. Ruiz-García, *FEMA 356* Nonlinear Static Procedure – Proposed Improvements for C1, C2 and C3 (ATC 55 draft working paper), Stanford University, 2002c.