ATC-114 Next-Generation Hysteretic Relationships for Performance-based Modeling and Analysis

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Abstract

Nonlinear analysis has become an increasingly useful and important tool for evaluation, upgrade and design of structures for seismic resistance. However, despite steady improvements in analysis capability, most practice remains anchored to guidelines developed more than 20 years ago. Under its ATC-114 project, the Applied Technology Council is developing updated hysteretic envelope models for use in seismic analysis of new and existing buildings. The intent of this project is to support the development of updated building code criteria contained in such standards as ASCE 7 and ASCE 41. Project support is provided by the National Institute of Standards and Technology.

Introduction

Although researchers have been performing nonlinear analysis for many years, the first direct, practical applications of this technique in seismic design occurred in the 1980s, as engineers began to design base isolated structures. In these early applications, nonlinearity was limited to the base isolation system itself, with the superstructure modeled elastically, and often as a simple stick system having appropriate mass and modal properties. There are a number of reasons nonlinear analysis was not commonly used by engineers at this time including limited availability of software to perform such analysis and the fact that typical computers available to engineers did not have the required speed or processing capability necessary to make this technique practical. Another major impediment was the lack of consensus guidance or code-specified criteria on how to use nonlinear analysis in design. The first such guidance was developed by the SEAOC Seismology Committee in an Appendix to its Blue Book (SEAOC, 1990). This guidance, adopted in the *1991 Uniform Building Code* (ICBO, 1991) addressed only analysis of base isolated systems, as described above.

More general nonlinear analysis guidelines for use in seismic analysis and design were first developed as part of the ATC-33 project and appeared in the *FEMA 273/274 Guidelines and Commentary* (ATC, 1997). The ATC-33 project team recognized that a key problem faced in the design of seismic upgrades is to assure deformation compatibility between the existing building elements, often having limited ductility, and new retrofit elements. It was clear that the linear approach used for the design of new buildings, where deformation compatibility could be assured by detailing requirements, could not be relied upon for existing buildings, as detailing of the existing structure could not be controlled or modified. Instead analysis conducted in support of retrofit design would have to directly account for the nonlinear deformation capacity of the individual elements, old and new, and also compute demand deformations, considering the nonlinear behavior of the overall structural system.

The ATC-33 project team recognized that nonlinear dynamic analysis was not yet practical as a design tool, given the previously described limitations. Therefore, the team developed the *FEMA 273/274* guidelines around the concept of nonlinear static, or pushover, analysis. Recognizing that even pushover analysis would be a large step for the practicing engineer of the time, many of whom still relied on equivalent lateral force analysis in their design of new structures, the ATC-33 project included linear analysis procedures for structures with good regularity and limited ductility demand. The *FEMA 273/274* guidelines and commentary permitted the use of nonlinear dynamic analysis, recognizing that technique might someday become practical, but included many warnings to the engineer about the potential pitfalls associated with this technique and the need for both caution and external, expert review.

Following publication of *FEMA 273/274* and the closely related *ATC-40* (ATC, 1996) methodology, structural engineering applications developers recognized the power of the performance-based design approach embodied in these documents and began to incorporate nonlinear analysis, both static and dynamic, into their suites of software. Importantly, first RAM PERFORM, then CSI ETABs and SAP incorporated the FEMA 273/274 hysteretic backbone data and nonlinear acceptance criteria directly into their element libraries. Armed with these powerful new tools, structural engineers rapidly embraced nonlinear analysis as a design tool, not only for evaluation and retrofit of existing structures, but also design of new structures. In 2002 the American Society of Civil Engineers (ASCE) incorporated the *FEMA 273* nonlinear dynamic analysis guidelines into its ASCE 7 Standard (ASCE, 2002). Then, in 2006, ASCE updated *FEMA 273/274* and published it as the ASCE 41 Standard (ASCE, 2006).

Today, many engineers use nonlinear dynamic analysis not only for evaluation and retrofit of existing buildings but also for design of major new structures and it has become the preferred technique for design of high-rise buildings in regions of high seismic risk. However, despite the extensive laboratory investigations into nonlinear behavior of wood, masonry, concrete and steel elements that have occurred in the past 20 years, with few exceptions, engineers performing nonlinear dynamic analysis continue to use the basic hysteretic relationships and acceptance criteria derived from those relationships, first developed by the ATC-33 project.

The National Institute of Standards and Technology (NIST), in its role as lead NEHRP agency identified the need to support further development of performance-based design as a priority.

The ATC-114 project is one of a series of projects sponsored by NIST to further this goal. The specific objectives of the ATC-114 project are to provide updated recommendations on hysteretic characterization of nonlinear structural behavior for use in performance-based seismic design. Like *FEMA 273/274*, it addresses all four major structural materials. The eventual ATC-114 report will be informational in nature and is primarily intended for use by members of the ASCE 7 and ASCE 41 committees as well as other groups developing seismic analysis guidelines and standards.

ASCE 41 Hysteretic Backbones

The nonlinear analysis procedures first developed in *FEMA 273/274* and carried forward into the *ASCE 7* and *ASCE 41* standards parse structural elements into two primary types: deformation-controlled and force-controlled. Force-controlled elements, by definition, have no appreciable ductility and are represented in analysis as elastic elements. Acceptable behavior of force-controlled elements is judged based on the amount of force-demand computed from analysis, and the margin provided against exceedance of a lower-bound estimate of strength.

Deformation-controlled elements have non-negligible ductility. For nonlinear static analysis, *ASCE 41* specifies that element nonlinear modeling conform to one of two idealized relationships, the most general of which is shown in Figure 1. In the figure, *Q* represents a generalized force quantity and *Δ* a generalized deformation.

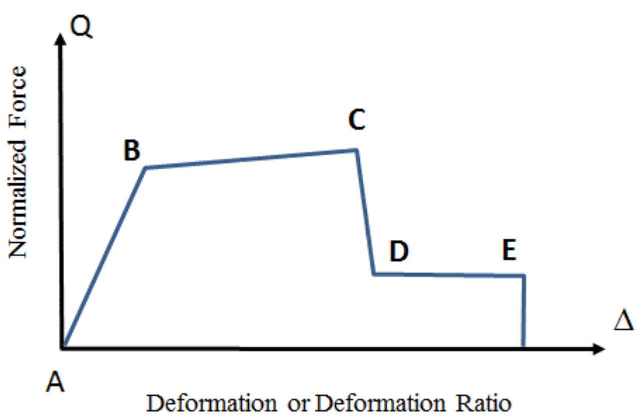

Figure 1 ASCE 41 Nonlinear Representation

The nonlinear behavior illustrated in Figure 1 was intended to represent, in an approximate manner, a second cycle backbone of data obtained from typical ramped cyclic component tests. The second cycle backbones, constructed as a series of secants drawn on hysteretic plots by connecting the crossings of the second cycle of loading to a given deformation increment with the first cycle for the next increment of loading. Second cycle backbones were intended to account for cyclic strength and stiffness degradation in an approximate manner, recognizing that static analysis could never accurately capture these effects. Figure 2 illustrates the construction of these second-cycle backbones from test data as well as the 1st cycle backbone later adopted by *ASCE 41*.

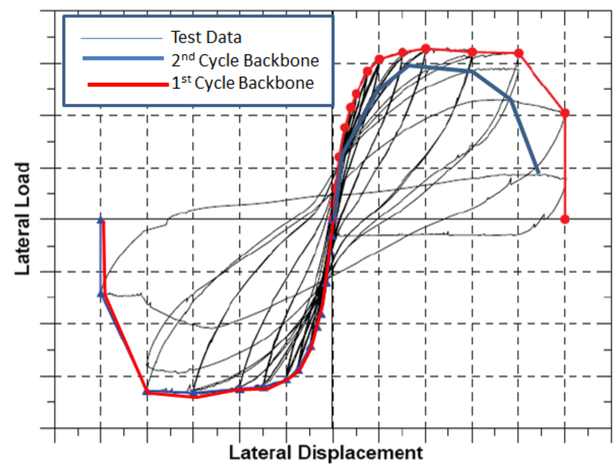

Figure 2 Development of cyclic backbones

FEMA 273/274 never intended the use of these backbones in nonlinear dynamic analysis and noted that hysteretic models for such analysis should be derived to match appropriate cyclic test data. However, lacking such data, engineers often applied the *FEMA 273/274* nonlinear representations, many of which were developed without specific data using judgment, for both static and dynamic analysis.

In 1997 FEMA entered into a cooperative effort with the American Society of Civil Engineers to develop *FEMA 273/274* into a pre-standard, *FEMA 356* (ASCE, 2000). *FEMA 356* carried forward nearly all of the *FEMA 273/274* hysteretic models with the exception that modeling recommendations for steel moment frames were updated using data available from the recently completed FEMA/SAC program to address seismic hazards in welded steel moment frames. *ASCE 41*, published in 2006, retained nearly all of the hysteretic models contained in *FEMA 356*, however, supplement 1 to the new standard, published in 2007 contained updates to the models for reinforced concrete columns. The 2013 edition of the standard (ASCE, 2013) revised the definition of the standard hysteretic model from a second cycle to first cycle backbone. As illustrated in Figure 2, the first cycle backbone typically exhibits less degradation than does the second cycle backbone, and as a result provides both greater deformation capacity and strength retention at large ductility demand. The argument used to support this change is that structures responding to ground motions with the low probability of exceedance associated with structural collapse often experience only a few large nonlinear excursions, and that the many cycles represented by second cycle curves over-stated the degradation experienced by such structures. Regardless of this definition change, the values of the control points (A, B, C, D, and E in Figure 1) specified by the standard for most elements did not change.

In addition to defining the force-deformation relations used to model nonlinear behavior in analysis the *ASCE 41* backbones also form the basis for acceptance criteria for deformation controlled components. Figure 3 illustrates the relationship of nonlinear deformation acceptance criteria to points on the backbone for primary and secondary components. Acceptance criteria used for linear analysis are generally take as 75% of those permitted for nonlinear analysis, accounting in approximate manner for the greater inherent uncertainty associated with demands predicted by linear analysis.

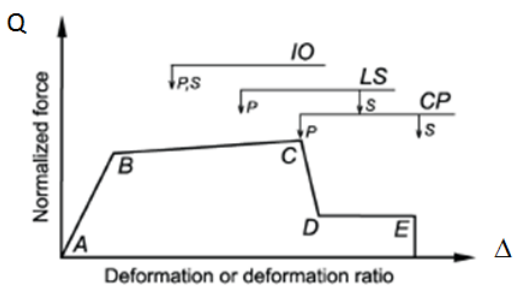

Figure 3 – Deformation acceptance criteria

The *ASCE 41* hysteretic models are largely based on laboratory testing that was available in the mid-1990s when the ATC-33 project was conducted. For many of the component types covered by the *ASCE 41* standard there was no such data available and therefore, the ATC-33 project team relied on judgement to develop these models. In the time since a large body of additional laboratory testing has become available. One important purpose of the ATC-114 project is to provide improved recommendations for hysteretic models based on this updated data. A second purpose is to update the basic ASCE 41 model based on the improved current understanding of nonlinear analysis and behavior now available.

The ASCE 41 backbones, even when based on laboratory data, are lacking in two primary ways. First, the backbones are inherently tied to the cyclic protocol on which they are based. Second, the backbones, though intended to represent best estimate, or median relationships provide no indication of potential variability or uncertainty.

Loading Effect on Hysteretic Response

Starting in the 1970s most laboratory testing of components conducted to characterize likely seismic behavior used ramped cyclic protocols in which the specimen was subjected to repeated fully reversed cycles of motion to increasing deformation amplitude such as that illustrated in Figure 4. Protocols such as that contained in ATC-24 (ATC, 1992) used so-called rain flow analysis to attempt to balance both the hysteretic energy and number and size of inelastic excursions contained in such protocols against that likely to be experienced by a structural component in a severe earthquake, however, response plots for elements obtained from nonlinear analysis seldom look much like these testing protocols. Often, as illustrated in Figure 5, response to real earthquakes entails relatively few large cycles of motion, early in the record, followed by a large number of smaller cycles, often about a permanently displaced position. Hysteretic backbones derived from ramped cyclic protocols may not accurately represent behavior in real earthquakes.

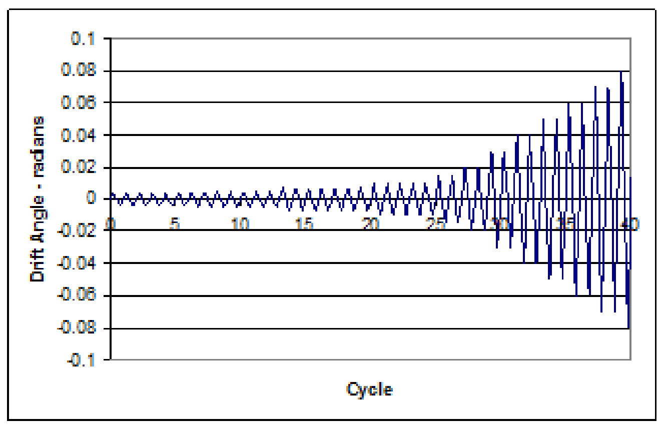

Figure 4 – Ramped cyclic testing protocol

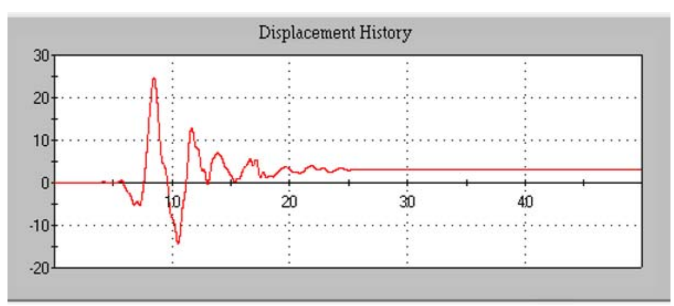

Figure 5 – Displacement History, 1 second structure, LGP000, 1989 Loma Prieta earthquake

The hysteretic response of some structural elements is significantly affected by the loading protocol used. Figure 6, taken from Suzuki and Lignos (2015) illustrates the difference in nonlinear response for a steel column loaded using the typical ATC-24 protocol, a monotonic loading and an alternative cyclic loading termed a collapse protocol that entails relatively few cycles and an increasing directional bias associated with onset of P-delta instability.

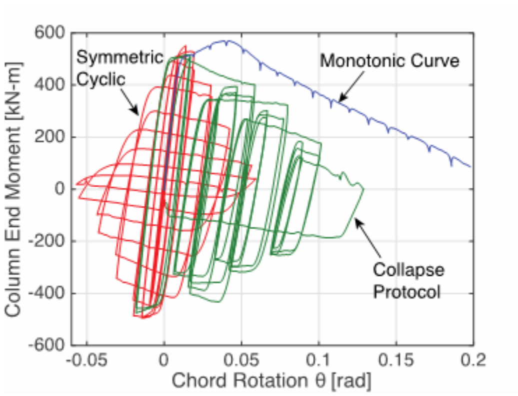

Figure 6 – Response of steel beam to alternate loading protocols

As can be seen in the figure, the monotonic response is characterized by increased strength, delayed onset of degradation and substantially larger ultimate deformation capacity than either of the cyclic protocols. Response to the so-called "collapse protocol" though it incorporates substantial cyclic degradation, demonstrates substantially increased deformation capacity than does response to the ATC-24 protocol loading. This response is consistent with findings from recent shake table collapse experiments on steel frame buildings (Lignos et al. 2011; Suita et al. 2008).

Since the response induced in structures by each earthquake ground motion is unique, no one loading protocol, nor any hysteretic model based on a single protocol can accurately capture a structure's likely response to the wide range of motions it may be subject to. Ideally, hysteretic behaviors incorporated in structural elements used for nonlinear analysis should be adaptive, and be able to accurately reproduce the response of elements to any testing protocol applied. A few such adaptive hysteretic models presently exist, though none are compatible with software commonly employed in design offices. However, the ATC-114 project team anticipates that such elements will become increasingly available. To facilitate the future development of such models, the ATC-114 team elected to present both cyclic and monotonic hysteretic models, for each component type, where sufficient monotonic data is available to permit this. The resulting data is presented in the format illustrated in Figure 7. In the figure, the blue curve represents monotonic response while the red curve response to a typical cyclic protocol, such as that of ATC-24.

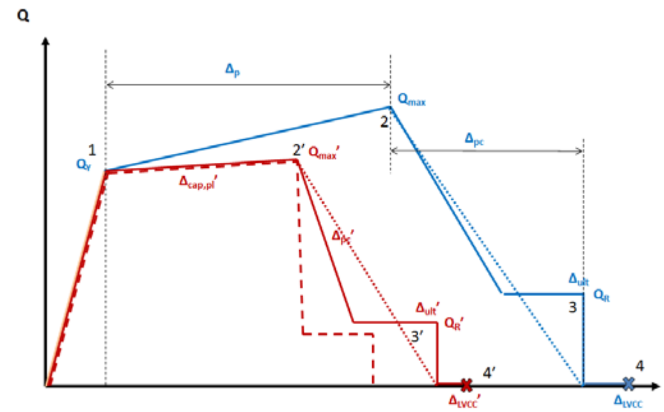

Figure 7 ATC 114 hysteretic models

Key parameters on the monotonic backbone include the yield point Q_y, Δ_y, which is typically protocol independent; the peak strength point Q_{max}, Δ_m; the post-capping strength point Q_R, Δ_{pc}; the point of loss of lateral force resistance Q_R, Δ_{ult}; and the loss of gravity (vertical) load resistance, Δ_{LVCC}. Control points for the cyclic backbone have similar designations with a prime (') denotation. It should be noted that for a number of reasons, few laboratory tests explore specimen behavior out to Δ_{LVCC} or even Δ_{ult}. Therefore, each of the ATC-114 recommended hysteretic models includes a recommended limit on the valid range of modeling, i.e. the deformation level at which behavior is not known simply because no test data is available to show behavior at larger deformation levels.

For both cyclic and monotonic backbones, two paths are shown beyond the peak load strength points, Q_{max}, Q'_{max}. One path includes a residual strength plateau, Q_R, Q'_R, and the other does not. The path with residual strength is used only for those component types that have a specific post-initial failure behavior that corresponds to the residual strength. An example is a simple shear plate connection attaching a beam to a column and supporting a slab. When loaded with the slab in compression initial behavior will be a composite of the steel clip plate and connecting bolts in tension and the slab in compression. An initial failure will consist of crushing of the slab, resulting in loss of the composite action with behavior dominated by the flexural capacity of the shear plate and bolts. This is illustrated in Figure 8, taken from Liu and Astaneh (2008) in which the positive quadrant is characterized by initial composite behavior, lost after crushing while the negative quadrant is limited to the flexural behavior of the shear plate. In many other component types, no such physical behavior, associated with development of a residual strength exists, and the apparent residual strength observed in plots of test data is really a consequence of the large increments in loading taken at large deformations. The steep decline found in ASCE 41 backbones for many component types is not believed to be real for most component types and can result in analytical instability. Therefore, the more gradual degrading phase is adopted by ATC 114 for many such component types.

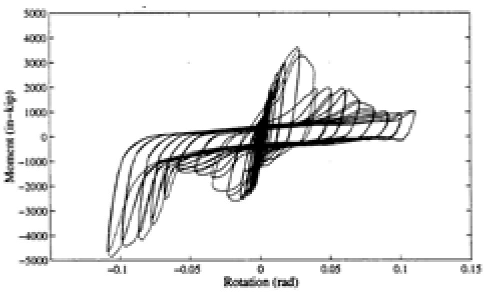

Figure 8 Hysteretic behavior of steel clip plate connection with composite slab (Liu and Astaneh, 2008)

Since monotonic backbones are always going to be an upper bound on an element's strength and deformation capacity they should never be used directly for seismic analysis. Rather the intent is that these monotonic backbones can be used by element developers to calibrate their adaptive models. Adaptive hysteretic models are the preferred approach for nonlinear dynamic analysis as they have the potential to most accurately portray true response, regardless of the particular ground motion's loading path. However adaptive models should not be used for nonlinear static analysis as it is impossible for static analyses to replicate any specific loading history, and therefore allow an adaptive model to capture correct stiffness, strength or deformation capacity. Rather, the cyclic model should continue to be used for nonlinear static analysis.

Modeling Uncertainty

Since the introduction of Load and Resistance Factor design methods, design procedures for new construction have been formulated to achieve target reliabilities with both load and resistance factors established considering uncertainties in load intensity, analytical methods and capacities. *FEMA 274* commentary describes an intended low probability, suggested as on the order of 10%, that a structure upgraded to meet specific performance objectives would fail to do so when actually subjected to a design event. However, *ASCE 41* does not have design procedures or criteria established to specifically achieve such reliability. Rather, the ASCE 41 procedures handle reliability in a qualitative manner. Specifically, the backbone control points are intended to be median values. When definition of the structure's construction obtained from drawings, specifications and field investigation is limited, a knowledge factor is applied to discount acceptance criteria. Lower bound values, estimated at 5^{th} percentile, are used for acceptance criteria for force-controlled behaviors.

More recently, design procedures incorporating nonlinear analysis have used more rigorous statistical methods to achieve desired reliability. The design procedures recommended by *FEMA 350* (SAC, 2000) included rigorous incorporation of uncertainties in demand prediction, element capacity and global structural capacity in a demand and resistance factor format. This procedure parsed uncertainty into epistemic (uncertain, or reducible) and aleatory (random) parts to enable targeting of both *FEMA 350* a probability of failure and a confidence level associated with achieving that failure. Later, the *FEMA P695* (ATC, 2009) procedure simplified this computation by combining aleatory and epistemic uncertainties into a single quantity and establishing that the reliability goal for ordinary occupancy structures is to provide less than a 10% probability of collapse, given MCE shaking. The Pacific Earthquake Engineering Research Center applied a load factor approach in their procedures for performance-based design of tall buildings (PEER, 2009) to account for uncertainties in demand prediction, while relying on the use of lower bound values for acceptance criteria, in order to achieve the target 10% probability of collapse suggested by *FEMA P695*. *ASCE 7-16* (ASCE, 2016) expanded the PEER methodology to address acceptance criteria for both force and deformation-controlled components considering uncertainty in demand prediction and capacity.

To facilitate the development of appropriate demand and resistance factors in future editions of *ASCE 7*, *ASCE 41* and other design procedures, the ATC-114 project elected to provide coefficients of variation, to represent uncertainty in both the strength and deformation values assigned to the control points in the cyclic and monotonic backbones.

Acceptance Criteria

As noted earlier, and illustrated in Figure 3, ASCE 41 uses the hysteretic backbones as its basis for acceptance criteria for deformation controlled behaviors for both nonlinear and linear procedures. The nonlinear static procedure attempts to produce mean estimates of demand while the acceptance criteria themselves are mean values. Assuming that the uncertainty distribution around capacity and demand are both represented by lognormal distributions, and assuming dispersions in demand and capacity respectively on the order of 0.5 and 0.2, there is roughly a 40% chance that any element evaluated as just meeting the acceptance criteria under predicted demands would actually exceed the deformation associated with the acceptance criteria if the structure were subjected to design earthquake loading. This is not particularly compatible with the FEMA 274 stated goal of approximately a 90% reliability level, i.e. 10% chance of failure. Fortunately, only the Collapse Prevention performance level in ASCE 41 has real physical meaning- that is, the structure does not collapse. Most of the acceptance criteria for deformation-controlled behaviors associated with Collapse Prevention in ASCE 41 do not represent actual loss of gravity load carrying capacity, but instead, in some cases, loss of most of the element's lateral force carrying resistance, and in many cases, the end of predictable behavior, simply because testing upon which the acceptance criteria are based did not proceed beyond that deformation level.

The probability of designs conforming to ASCE 41 not experiencing failure of force-controlled behaviors is similarly poor. The acceptance criteria for force controlled behaviors in ASCE 41 are intended to be lower bound values, specifically, 5th percentile. The demands computed from analysis however, are mean values. Assuming uncertainties in the capacity equation on the order of 15% and uncertainties in demand prediction on the order of 40%, it can be seen that force-controlled components designed with demand to capacity ratios of unity will have approximately a 25% chance of failure, given the occurrence of design ground shaking, not a particularly desirable outcome.

ASCE 7-16 (ASCE, 2016) Chapter 16 recognizes this unacceptable failure probability inherent in the ASCE 41 approach and applies load and resistance factors to the acceptance evaluation equations so as to provide the desired 90% reliability for elements having computed demand to capacity ratios of unity, and superior reliability for elements with lower design demand to capacity ratios.

By providing uncertainty values for the hysteretic backbone control points, ATC-114 makes it possible to apply a similar approach to the *ASCE 41* evaluation procedures. However, the problem remains that most laboratory tests stop at deformation levels below those that actually cause loss of gravity load carrying capacity, and, as a result, acceptance checks are based on presumed failure at the limit of available laboratory data and will be more conservative than if actual failure data were available. Further, to the extent that a real earthquake loading may not result in as much degradation as that experienced in ramped cyclic laboratory testing, the acceptance criteria will have greater conservatism. While this may be a reasonable situation, given that failure is undesirable, the use of excessively conservative acceptance criteria will result in retrofit designs that are needlessly costly, and may result in some structures remaining vulnerable because the cost of retrofit was excessive.

The use of models incorporating elements with adaptive hysteretic relationships can reduce part of this conservatism by providing demands that have not been biased by excessive degradation in the response model. However, as illustrated in Figure 9 it is not clear how to develop acceptance criteria based on such adaptive models. In the figure, the standard cyclic backbone is shown in red, together with the control points associated with acceptance criteria, for primary and secondary deformation-controlled components. The collapse prevention acceptance criteria for deformation-controlled components at Δ_{LVCC}. The blue curve is the monotonic envelope, for which no acceptance criteria are shown because all earthquakes include some inelastic cycles and will result in some cyclic degradation.

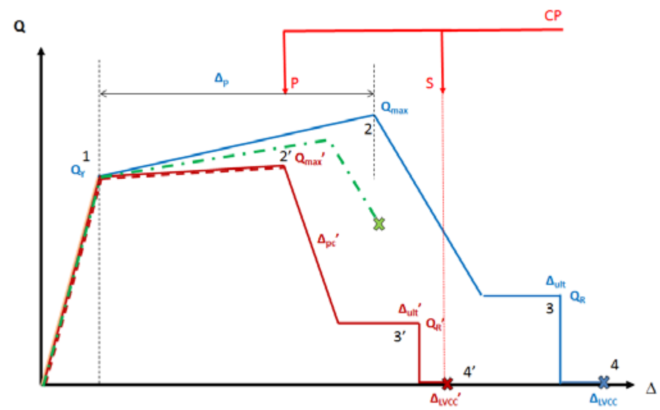

Figure 9 Acceptance criteria and hysteretic backbones

In the figure, the green curve represents the envelope of adaptive hysteretic response for an element responding to a hypothetical earthquake motion. As will typically be the case, the adaptive backbone shows less degradation than does the cyclic backbone and more than the monotonic envelope. However, the exact shape of these adaptive response envelopes will be unique to the ground motion and structure. Further, the end point for the green adaptive envelope does not represent a capacity, but is simply the maximum deformation experienced in response to the particular ground motion.

Since it is not possible to develop collapse prevention acceptance criteria based on the adaptive hysteretic models, it is proposed instead that when adaptive models are used, a run be considered to produce acceptable response as long as the analysis converges, and demands on force-controlled components remain within acceptable limits. In order to gain confidence that failure will not result when this approach is taken, it will be necessary to perform analyses using a sufficiently large suite of motions to obtain statistics on the probability of collapse. For example, ASCE 7-16 requires a suite of 11 motions so as to gain moderate confidence (75%) that collapse probability if less than 10%. Larger suites of analyses are necessary to gain higher confidence.

When models are constructed using elements tuned to cyclic backbones only, or when nonlinear static analysis is performed, acceptance criteria for deformation-controlled behaviors should continue to be based on the backbone control points illustrated in Figure 9, with the exception that uncertainty should be considered in judging acceptance, as done in ASCE 7.

The sections below present summary level recommendations for steel braced frames and moment frames, flexure-governed concrete walls, reinforced masonry walls and wood walls. Work on shear-controlled concrete walls and concrete moment frames is under development and will be included in the final ATC-114 report.

Steel Concentric Braced Frames

For special concentric braced frames (SCBFs) complying with *AISC 341-10* (AISC, 2010) explicit modeling of the braces and gussets in a manner that will directly capture cyclic degradation due to buckling is recommended. To accomplish this, individual braces should be modeled with a minimum of four elements per brace extending between connections. The geometry of the brace elements should be configured such that an out-of-plane off set at brace mid-point with an amplitude of 1/500 of brace length is captured. Beam column joints should be modeled with rigid end offsets for the brace, beam and column, and a rotational spring representing the gusset plate stiffness. Figure 10 shows the recommended modeling approach at a typical brace-beam-column connection and Figure 11 illustrates the hysteresis obtained from typical models of this type.

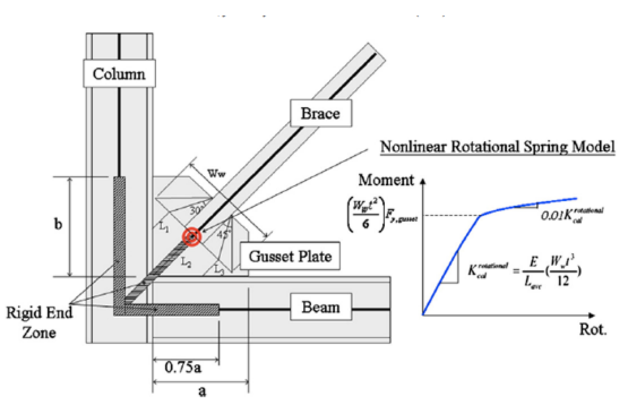

Figure 10 – Model of brace to beam and column connections

For braces conforming to the criteria for SCBFs failure and loss of strength will result from brace fracture due to low-cycle fatigue and the large plastic strains that occur at the point of plastic hinge formation during buckling. For cyclic loading protocols, the value of the brace section slenderness ratio, either h/d or b/t, is the best predictive parameter for failure. Figure 12 presents the recommended cyclic deformation limit, measured as frame story drift ratio, as a function of the ratio of the critical section slenderness parameter, λ to the limiting value λ_{hd} specified by AISC 241-10 for highly ductile elements.

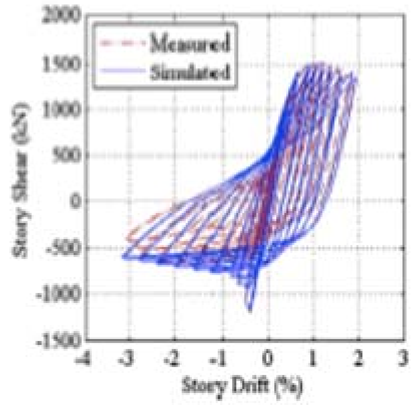

Figure 11 Comparison of actual and analytically predicted hysteresis for adaptive braced frame model

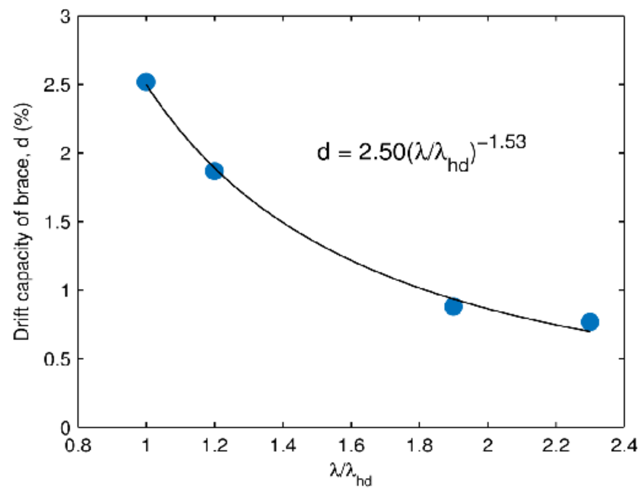

Figure 12 Limiting deformation of SCBF brace as a function of section slenderness

For nonlinear static analysis, it is recommended that pairs of braces (one in tension and one in compression) be represented by a single element having the simplified backbone illustrated in Figure 13. In the figure Q_{ave} is the average of the brace yield and compressive strengths, Δ_{ave} is the average of the deformation at initiation of yield and buckling and f is the residual strength at fracture given by the equation shown in the figure. Deformation at failure is obtained from Figure 12, just as for nonlinear dynamic analysis.

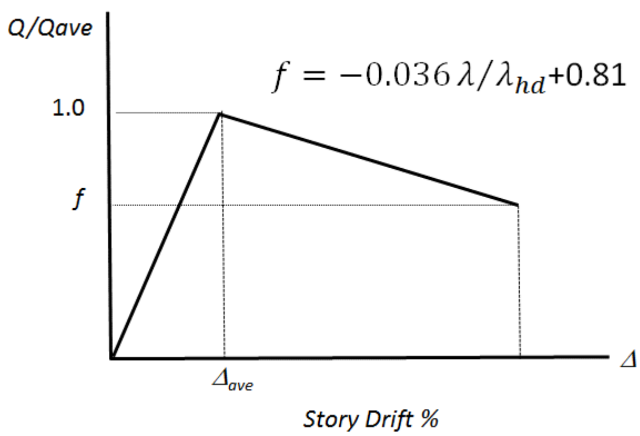

Figure 13 SCBF cyclic backbone showing residual strength

Flexure-controlled Concrete Walls

Planar reinforced concrete shear walls controlled by flexure can be adequately modeled using fiber-type beam-column elements. Figure 14 illustrates the cross section of such a beam-column element and Figure 15 the constitutive properties for the concrete and steel fibers. Figure 16 illustrates a comparison of three different such models with the cyclic test for the modeled wall. Modeling accuracy can be improved with the addition of nonlinear shear models such as that shown in Figure 17.

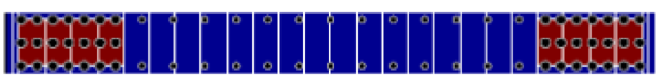

Figure 14 Cross section of fiber-type beam column element

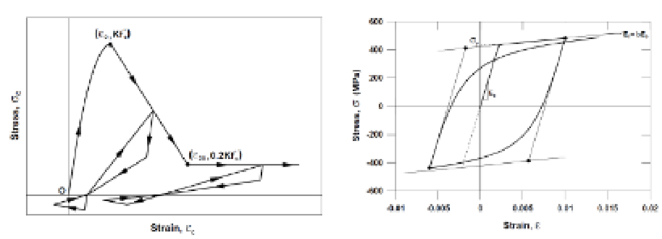

Figure 15 Constitutive models for concrete and steel materials

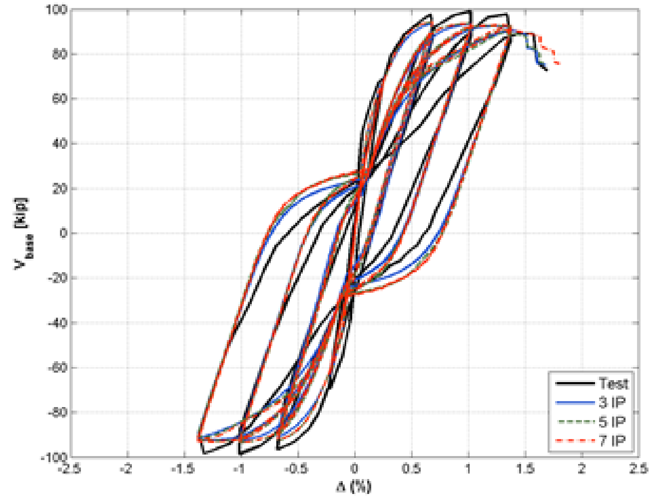

Figure 16 Measured and analytically predicted response of planar wall using three alternative models

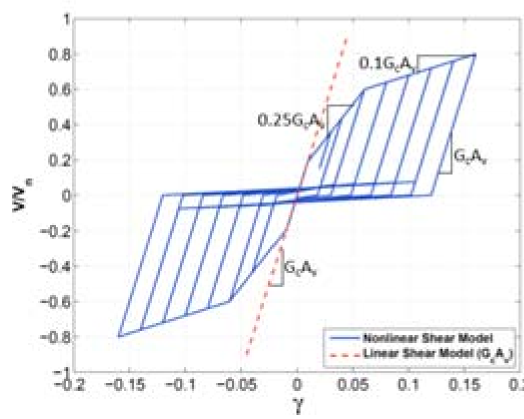

Figure 17 – Nonlinear shear model for use with flexural beam-column wall models

Based on work by Whitman (2015) flexural behavior of concrete shear walls can be viewed as controlled either by shear-compressive interaction, buckling compression failure, or buckling-induced rupture of bars. As illustrated in Figure 18, the controlling mode can be predicted as function of shear stress and cross-sectional aspect ratio. Limiting plastic hinge rotations of 1.6% and 0.9% are recommended respectively for walls controlled by compression buckling or buckling rupture, and compressive shear interaction failure.

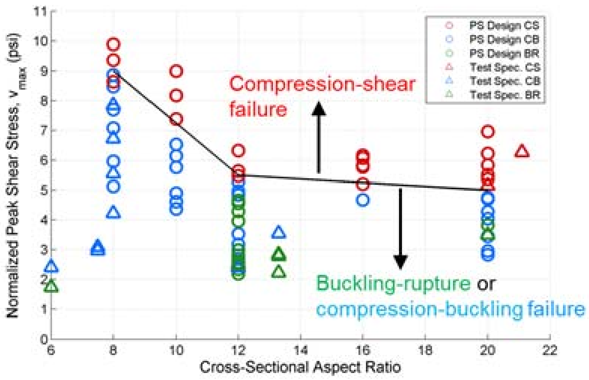

Figure 18 Concrete wall flexural failure as a function of shear stress and cross section aspect ratio

Reinforced Masonry Walls

Reinforced masonry walls should be modeled as assemblages of beam-column elements calibrated to reflect the excepted plastic-hinge length of the wall components. The shear behavior of a wall component should be represented by a linear or nonlinear shear spring. Cyclic backbone curves for flexural behavior are presented based on detailed analytical models calibrated to 21 cyclic tests of walls employing different aspect ratios, reinforcement ratios and axial loading. Similarly, cyclic backbone curves for shear behavior are presented based on similar analytical modeling calibrated to 16 cyclic tests. These walls were all fully grouted. Monotonic tests are not available for reinforced masonry elements.

Depending on the aspect ratio, axial loading and reinforcement ratio, masonry walls dominated by nonlinear flexural response exhibit ductile hysteresis, such as that shown in Figure 19. The pinching and eventual load degradation exhibited in the hysteresis are normally caused by masonry crushing, rebar buckling and fracture, or the failure of the lap splices in the vertical reinforcement.

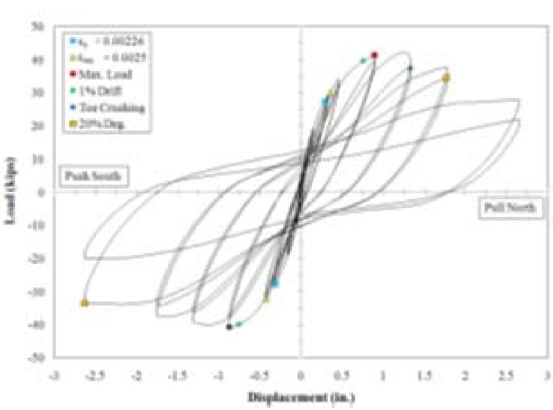

Figure 19 Typical flexural hysteresis for reinforced masonry wall (Sherman, 2011)

Figure 20 illustrates the recommended cyclic backbone for reinforced masonry walls. Key control points are the point at which 80% of peak strength is achieved upon initial loading, the point of peak strength, and points at which strength degrades to 75% and 50% of peak values, the latter representing the end of the valid range of modeling. Displacements at these key control points are presented as a function of axial compression ratio and reinforcing ratio.

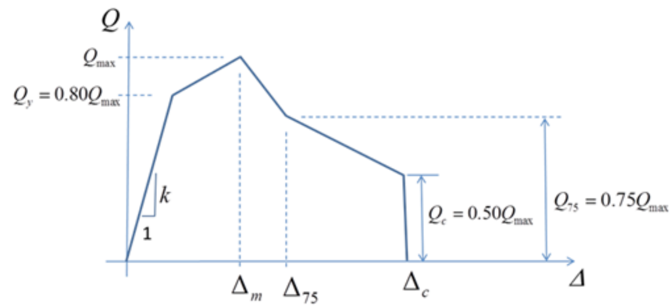

Figure 20 Cyclic backbone for flexure-controlled reinforced masonry walls

Figure 21 presents the recommended cyclic backbone for reinforced masonry walls with shear nonlinearity. Peak shear strength is assumed to occur at a shear drift ratio of 0.5%, degradation to residual strength at twice that deformation and the valid range of modeling is assumed at 2% drift. The residual strength is a function of the shear reinforcing ratio as given by equation (1). These values are based on test data for full grouted walls. Partially grouted walls will exhibit more brittle behavior. When nonlinear dynamic analysis is performed hysteretic behavior should include significant pinching, similar to that indicated in Figure 19 for flexural-controlled walls.

$$Q_r = V_{ns} = 0.5\frac{A_v}{s} f_y d_v \qquad \text{Eq. 1}$$

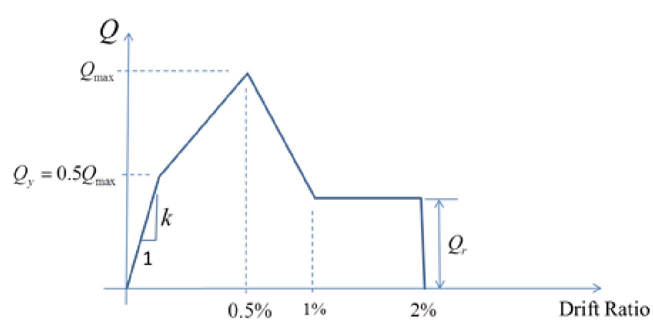

Figure 21 Cyclic backbone for shear-controlled reinforced masonry walls

Wood Shear Walls

Wood shear wall hysteretic response is characterized by a lack of a well-defined yield point and continuous softening from initial loading through achievement of peak resistance followed by degradation. When nonlinear dynamic analysis is performed the CUREE element shown in Figure 22 is recommended, however, any element with significant pinching and the ability to capture stiffness and strength degradation can be used.

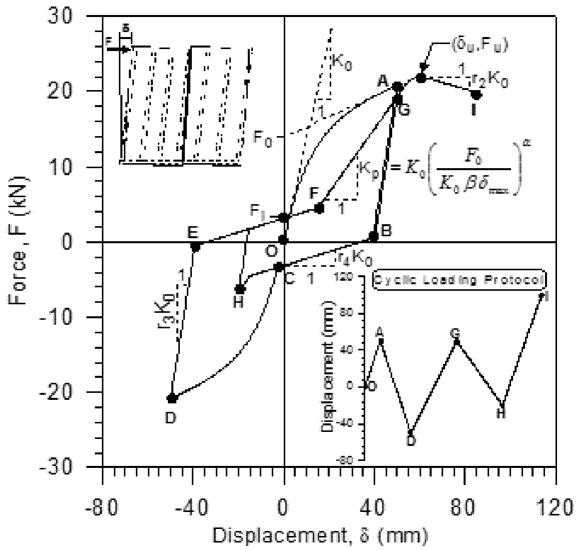

Figure 22 CUREE (Folz and Filliatrualt, 2001) wood wall model

The standard cyclic backbone shown in Figure 23 is recommended. The parameters, F_0, F_u, r_1, r_2, and $\Delta_{u,max}$ are defined in tabular form dependent on the sheathing material, nailing and wall aspect ratio.

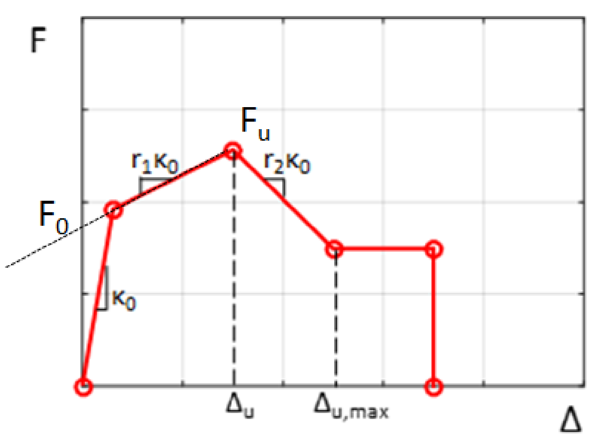

Figure 23 Cyclic backbone for wood walls with various sheathing materials

Steel Moment Resisting Frames

The extensive testing of steel frame components of various types conducted following the 1994 Northridge earthquake have enabled the development of robust, adaptive models. These models have been used to develop recommended cyclic and monotonic backbones together with the key control parameters for a variety of beams, beam-columns and beam column connections, considering section compactness, lateral bracing and connection type. As illustrated in Figure 24, steel beams with RBS (Ricles et al. 2004) subjected to monotonic loading exhibit substantially reduced degradation and additional ductility.

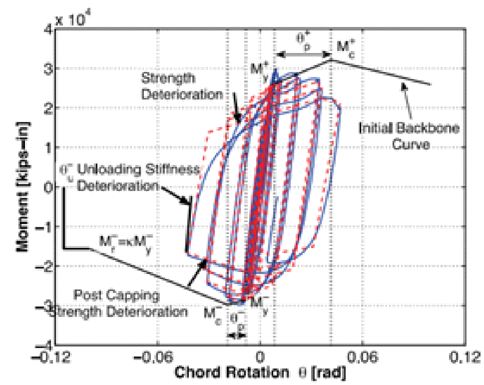

Figure 24 – Cyclic and monotonic loading behavior for RBS beam to column connection

For steel elements, cyclic and monotonic backbones like those shown in Figure 7 are presented. Depending on the steel structural element, control points on their backbones are a function of axial load ratio, h/t_w, b_f/t_f, L/r, and *L/d*. Equation 2, below illustrates the form of equations provided to define these control points, this one presented for a fully-restrained beam to column connection, without reduced beam section. In the equation, the parameter c_{unit} is a conversion factor between metric and English units. Coefficients of variation are provided for all parameters.

$$\theta_{cap,pl} = 0.07 \cdot \left(\frac{h}{t_w}\right)^{-0.3} \cdot \left(\frac{b_f}{2t_f}\right)^{-0.1} \cdot \left(\frac{L}{d}\right)^{0.3} \cdot \left(\frac{c_{unit} d}{533}\right)^{-0.7} \quad \text{Eq. 2}$$

Importantly, based on recent testing (Suzuki and Lignos 2015; Elkady and Lignos 2016) permissible ductility for steel columns with substantial axially load is greatly increased relative to the values contained in ASCE 41. Further, recommendation is made to base the axial load ratio on sustained, gravity load, rather than peak transient load.

Summary

Upon completion, forecast for the spring of 2017, the ATC-114 project will be a valuable resource to code and standards committees engaged in the development of criteria for using nonlinear analysis in design and retrofit, as well as individual engineers engaged on projects. The completed document will provide greatly improved backbone relationships for structures of concrete, masonry, steel and wood construction. These backbones will permit improved modeling and also, in many cases, less restrictive acceptance criteria. In the ASCE 41 standard, where acceptance criteria for linear procedures are derived from the backbones this will also provide benefit for the linear procedures.

For steel frame buildings, adaptive models that are capable of representing hysteretic response without calibration to specific loading protocols are available. As these models find their way into software commonly used in the design office, and as other similar models are developed for other structural systems, nonlinear dynamic analysis will become more attractive on design projects as the behavior of structures subjected to extreme loadings will be better predicted, allowing liberalization of the acceptance criteria presently available.

Though not specifically addressed herein, two companion projects being conducted under the ATC-114 contract address detailed modeling and analysis criteria for structural steel and reinforced concrete moment frames. These companion publications will be immediately useful to engineers performing nonlinear analysis of these structures in design and evaluation projects.

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