

# LOADING HISTORIES FOR CYCLIC TESTS IN SUPPORT OF PERFORMANCE ASSESSMENT OF STRUCTURAL COMPONENTS

Helmut Krawinkler<sup>1</sup>

## ABSTRACT

The need for comparable loading histories for seismic acceptance testing is becoming more prevalent as (a) performance-based design, which requires quantification of performance, is becoming a more widely accepted alternative to routine code design, (b) more and more innovative performance enhancement systems (e.g., buckling restrained braces) hit the engineering market, and (c) globalization becomes a widely accepted concept that necessitates globally accepted performance standards. Present codes, standards, and guidelines make reference to the need for performance assessment through testing, but with few exceptions (e.g., AISC 341-05 and testing of base isolation systems in ASCE 7-05) they remain mostly silent on testing and acceptance criteria to be used for this purpose. This paper provides a summary of (a) the background behind several of the presently employed cyclic loading histories, (b) basic concepts that should be considered in the development of loading histories, and (c) recent developments that point towards the need for modifications or expansion of presently employed loading protocols.

## WHY DO WE HAVE TO BOTHER WITH LOADING PROTOCOLS

All structural elements have limited strength and deformation capacities; and collapse safety as well as damage control depend on our ability to assess these capacities with some confidence. Implicitly, we lump our knowledge of these capacities into response modification coefficients ( $R$ -factors) for new structures (ASCE 7-05), and into  $m$ -factors (or estimates of plastic deformation capacities) for seismic retrofits (FEMA 273/356 and ASCE 41-06). For some cases our knowledge is adequate to assign reasonable values, for many cases it is not. So, we have to resort to testing (in addition to analytical modeling) to evaluate performance of many conventional components, and particularly of new and innovative components (or systems) that show much promise for enhanced performance. Unfortunately, in earthquake engineering, strength and deformation capacities depend (sometimes weakly and sometimes strongly) on cumulative damage, which implies that every component has a permanent memory of past damaging events and at any instance in time it will remember all the past excursions (or cycles) that have contributed to the deterioration in its state of health. Thus, performance depends on the history of previously applied damaging cycles, and the only reasonable way to assess the consequences of history (short of developing complex analytical models that can be used for damage state predictions) is to replicate, to the best we can, the load and deformation histories a component will undergo in an earthquake (or several earthquakes if this is appropriate). The objective of a loading protocol is to achieve this in a conservative, yet not too conservative, manner.

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<sup>1</sup> Dept. of Civil and Environmental Engineering, Stanford University; Stanford, CA

The need for representative loading histories is becoming more prevalent as performance-based seismic design, which requires quantification of performance, is becoming a more widely accepted alternative to routine code design, and as more and more innovative performance enhancement systems become available. Present codes, standards, and guidelines make reference to the need for performance assessment through testing in various sections. For instance, ASCE 7-05 states in Section 12.2.1, “Seismic force-resisting systems that are not contained in Table 12.2-1 are permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 12.2-1 for equivalent response modification coefficient,  $R$ , system overstrength coefficient  $\Omega_0$ , and deflection amplification factor,  $C_d$ , values.” However, except for testing of isolators and dampers for verifying the properties used in design (Sections 17.8 and 18.9), this standards document remains silent on testing and acceptance criteria to be used for this purpose.

### **CONCEPTS BEHIND DEVELOPMENT OF LOADING PROTOCOLS**

There is no unique and “best” loading history, because no two earthquakes are alike and because the specimen may be part of many different structural configurations. The overriding issue is to account for cumulative damage effects through cyclic loading. If there is no cumulative damage, there is no need for cyclic loading. The number and amplitudes of cycles applied to the specimen may be derived from analytical studies in which models of representative structural systems are subjected to representative earthquake ground motions and the response is evaluated statistically.

Any loading protocol will always be a compromise that will provide deformation histories whose realism will depend on many parameters. For one, actual histories, as experienced in earthquakes, will depend on the intensity and frequency content (magnitude, distance, and soil type dependence) of the ground motion the specific component (or assembly) will be subjected to as part of the structural system. Equally important, the number and amplitudes of cycles the component will experience depend on the configuration, strength, stiffness, and modal properties (periods and participation factors) of the structure and on the deterioration characteristics of the structural systems and its components. Moreover, in most practical cases (except for near-fault ground motions) the actual sequence of cycles (excursions) as experienced in an earthquake has to be rearranged into cycles of increasing magnitude in order to avoid commitment to a single maximum amplitude that may have meaning only for a specific combination of ground motion and structural configuration.

For reasons just quoted, there are many decisions and judgmental steps to be taken in order to come up with a compromise loading history that is conservative but statistically representative of the full range of ground motions and structural configurations. These decisions and steps are too elaborate to enumerate here. The reader is referred to the following references for detailed discussions: ATC-24, 1992, Krawinkler et al., 2000-a, and Krawinkler et al., 2000-b. The overriding issue is that cumulative damage concepts (of the type presented in Krawinkler et al., 1983) have to be employed in order to guide the decision process for loading history development.

## EXAMPLES OF LOADING PROTOCOLS USED PRESENTLY OR IN THE RECENT PAST

Many loading protocols have been proposed in the literature, and several have been used in multi-institutional testing programs (e.g., ATC 1992, Clark et al. 1997, Krawinkler et al. 2000), or are contained in standards or are proposed for standards (e.g., FEMA 2007, AISC 2005, ASTM 2003, ICBO-ES 2002). These protocols recommend somewhat different loading histories, but in most cases they differ more in detail than in concept.

Figure 1 presents a series of loading protocols that have been developed and employed during the last 20 years. There is no claim for completeness, and no claim that the presented protocols are superior to others not mentioned here. It is interesting to note that most of the presented protocols are material specific (primarily steel and wood), and have not found application for materials other than those mentioned. Several of the protocols are for wood, which indicates that for wood assemblies, more so than for other materials, experimental methods are employed to determine acceptance for use in seismic construction. One reason may be that for this material analytical methods for performance prediction are rather unreliable, but probably the main reason is that it is cost-effective to test wood panels and connector in great numbers, whereas it is extremely expensive to perform many tests on components of steel or reinforced concrete structures. Thus, for economic reasons, seismic performance acceptance of wood components is based mostly on tests rather than analytical models. It is interesting to note that there appears to be no widely used loading protocol for components of reinforced concrete structures.

***Steel - ATC-24 Protocol (ATC-24, 1992), Figure 1(a).*** This protocol, which was specifically developed for components of steel structures, was one of the first formal protocols developed in the U.S. for seismic performance evaluation of components using a cyclic loading history. It uses the yield deformation,  $\Delta_{yield}$ , as the reference for increasing the amplitude of cycles. The history contains at least 6 elastic cycles (amplitude  $< \Delta_{yield}$ ), followed by three cycles each of amplitude  $\Delta_{yield}$ ,  $2\Delta_{yield}$ , and  $3\Delta_{yield}$ , followed by pairs of cycles whose amplitude increases in increments of  $\Delta_{yield}$  until severe cyclic deterioration occurs. The relative and absolute amplitudes of the cycles were derived from statistical studies of time history responses of SDOF systems, and therefore represent global (roof or story) drift histories and not local deformation histories such as those experienced, for instance, by links in eccentrically braced frames. [The ATC-24 document contains also a multi-specimen cumulative damage testing protocol that is based on constant amplitude testing and a cumulative damage hypothesis.]

This protocol was employed in Phase I of the SAC steel program. In the SAC Phase I experiments it was found that inconsistent measures of “yield deformation” were employed by different investigators (for test control,  $\Delta_{yield}$  had to be predicted before the test), which led to results that were difficult to compare with each other. Thus, the choice of a “yield deformation” as the test control parameter appears to be ambiguous even for steel (and certainly more so for materials such as reinforced concrete and wood). This discovery came too late to prevent adoption of this protocol in the 1997 AISC Seismic Provisions (Appendix S) for qualification testing of steel components.

***Steel - SAC Protocol (Clark et al., 1997), Figure 1(b).*** Because of the  $\Delta_{yield}$  ambiguity, and because of the opportunity the SAC program offered to develop a specific loading protocol for

steel moment frames, a statistical study was performed on the number and amplitudes of story drift cycles of the SAC Los Angeles and Seattle 3 and 9 story frame structures (Krawinkler et al., 2000-a). The outcome of this study is the SAC loading protocol shown in Figure 1(b), which uses story drift rather than yield deformation as the amplitude control parameter. For steel frame structures the story yield drift is confined to a rather narrow range around 0.01 radians, which permits an approximate correlation between the ATC-24 and SAC protocols. The SAC protocol contains more small (elastic) cycles (which were added because of the observed Northridge weld fractures that occurred before yielding took place), two cycles of an intermediate amplitude of 0.015 radians, but slightly fewer cycles of larger amplitude. In general the two protocols are very similar in cumulative damage potential, but because of the commitment to story drift as the control parameter, the SAC protocol should not be applied to configurations other than steel beam-to-column assemblies that are representative of typical stories. The November 2000 Supplement #2 of the 1997 AISC Seismic Provisions has replaced the ATC-24 protocol with the SAC protocol.

**Wood/Masonry – SPD Protocol (Porter, 1987), Figure 1(c).** This protocol was developed originally for the US-Japan TCCMAR Testing program for masonry research, with a focus on very rigid and short period structures. [It is not known to the writer whether this protocol shows up explicitly in any standards related to masonry structures.] In the late 90ies it was slightly modified and taken over by the CoLA committee for testing of woodframe shear walls. It distinguishes itself from the previous two protocols by the presence of many more cycles (small and large), and by the use of a “First Major Event” (FME) to control cycle amplitudes (SEAOSC, 2001). For wood structures, more so than for steel structures, the pre-test prediction of the FME is ambiguous to say the least. Thus, different laboratories may come up with quite different reference amplitudes (which is relevant only if the performance of test specimens have to be compared). The much more important difference is the much larger number of large deformation cycles in the SPD protocol. There is no doubt that this protocol imposes much more severe demands on a test specimen than either the ATC-24 or the SAC protocol. The potentially huge difference between the envelope of cyclic response curves and the corresponding monotonic response curve is illustrated in Figure 2(d). This protocol, which has been used extensively in the UC Irvine tests on plywood and OSB wall panel for the CoLA Committee, has led to many nail fatigue failures rather than nail withdrawal as was observed in most of the damaged panels after the Northridge Earthquake. Thus, for the type of damage observed in the Northridge earthquake, the SPD loading protocol is believed to be much too severe. The SPD protocol is one of the options included in the ASTM E 2126-02 standard and in ICBO-ES AC130-0102-0902.

**Wood – CUREE (Krawinkler et al., 2000-b), Figure 1(d).** Because of the discrepancies observed between Northridge earthquake and CoLA test damage, early in the CUREE-Caltech woodframe project a study was commissioned on loading protocols for woodframe shear wall testing. The outcome is the loading protocol shown in Figure 1(d). It appears to be close to the SAC loading protocol but has two clearly different features. First, the reference parameter for amplitude variation is neither a yield (or FME) deformation nor is it an absolute drift ratio (as in SAC). It is the maximum displacement,  $\Delta$ , for which the specimen is expected to exhibit acceptable performance when subjected to this loading protocol. It also could be a design target displacement for which the component is to be qualified. If the specimen performs satisfactory after execution of the first cycle with amplitude  $\Delta$ , then the specimen has passed the acceptance test at this target amplitude. If it fails the acceptance test at this amplitude, then one needs to

back up and decide whether it passes the acceptance test at the next smaller amplitude. If the specimen can sustain cycles with an amplitude larger than  $\Delta$ , then the test can be continued in the predetermined pattern (cycle with target amplitude followed by two smaller cycles, and then the amplitude increased to a new target value that is not permitted to be more than 50% larger than the previous target amplitude). Tests of this type avoid a commitment to an early event (yielding or FME), which then drives the loading history, but it necessitates commitment to a target amplitude for acceptable performance. Execution of a monotonic test to failure is most helpful for estimating this target amplitude. In (Krawinkler et al., 2000-b) it is suggested to use  $\Delta = 0.6\Delta_{\text{mon}}$  as an estimate.

There is one more clear difference between the CUREE protocol and the ATC-24 and SAC protocols, that being the presence of “trailing” cycles. These are the smaller cycles following the preceding larger (primary) cycle at each step. These trailing cycles do less (and often much less) damage than cycles of amplitude equal to the larger one. Time history analysis has shown that the pattern of larger cycles being followed by smaller cycles is indeed justifiable by statistical means. A simplified version of the CUREE protocol makes the amplitude of the trailing cycles equal to that of the preceding primary cycle – for simplicity only. The ASTM E 2126-02a standard includes the CUREE protocol as one of the loading options.

**Wood – ISO (ISO, 1998), Figure 1(e).** This protocol is illustrated because it is of foreign origin but uses concepts similar to those suggested in the CUREE protocol and is expected to provide similar acceptance levels. It uses the maximum displacement (called  $v_u$ ) as a reference value. Several small cycles are followed by three cycles each of amplitudes of 25%, 50%, 75%, and 100% (with further increments of 25% as needed) of the maximum displacement.

**FEMA 461 (FEMA 2007), Figure 1(f).** This protocol was developed originally for testing of drift sensitive nonstructural components, but is applicable in general also to drift sensitive structural components. It uses a targeted maximum deformation amplitude,  $\Delta_m$ , and a targeted smallest deformation amplitude,  $\Delta_0$ , as reference values, and a predetermined number of increments,  $n$ , to determine the loading history (a value of  $n \geq 10$  is recommended). The amplitude  $a_i$  of the step-wise increasing deformation cycles is given by the equation  $a_{i+1}/a_n = 1.4(a_i/a_n)$ , where  $a_1$  is equal to  $\Delta_0$  (or a value close to it) and  $a_n$  is equal to  $\Delta_m$  (or a value close to it). Two cycles are to be executed for each amplitude. If the last damage state has not yet occurred at the target value  $\Delta_m$ , the loading history shall be continued by using further increments of amplitude of  $0.3\Delta_m$ .

**Near-Fault Protocols: Steel – SAC (Krawinkler et al., 2000-a), Figure 1(g), and Wood – CUREE (Krawinkler et al., 2000-b), Figure 1(h).** The aforementioned loading protocols were developed without specific regards to near-fault ground motions with forward directivity. In both the SAC Steel and CUREE Woodframe projects, specific near-fault protocols were developed with the results shown at the bottom of Figure 1. The two protocols are not identical for two reasons. First, the CUREE protocol was developed for short period woodframe structures and the SAC protocol for medium to long period steel frame structures. The period range has a significant effect on the response to near-fault ground motions. Second, the SAC near-fault protocol contains a pulse reversal in its loading history (from cycle 8 on). The reason is that near-fault pulses can come from either direction, and in a steel beam-to-column connection may affect either the top or the bottom flange (SAC tests have shown that these two flanges show different

performance characteristics). Thus, the second part of the history serves to find out acceptability of performance of the connection when moments are applied in the opposite direction.

Both near-fault protocols have not found widespread application. One reason is the added testing expense (which may be huge for a test of steel or reinforced concrete assemblies), and the other is that there is some indication that the cyclic deterioration effect of the few cycles preceding the large pulse may be small enough so that monotonic test results can be used instead. But there is not sufficient evidence to make this statement with confidence – to the contrary, in some of the near-fault protocol wood tests the effect of the cycle with amplitude  $0.6\Delta_m$ , which precedes the one-sided displacement pulse, was not insignificant. It increased the specimen strength by a few percent but decrease the deformation capacity by about 17% in average (Gitto and Uang, 2002). Thus, a final verdict on the importance of a near-fault performance test is not yet in. Neither of the two protocols has yet found its way into standards.

### **Comparison of Results Obtained from Various Loading Protocols**

In the writer's opinion, the protocols presented in Figures 1(a), (b), (d), (e), and (f) are rather similar (because their energy dissipation demands are not much different) and are expected to produce similar performance assessments. The near-fault protocols in Figures 1(g) and (h) deserve more consideration unless it can be demonstrated that they produce performance that is closely represented by a monotonic test. This leaves the SPD protocol as the outlier (but being an outlier does not necessarily imply being "incorrect").

Differences in performance obtained from different loading protocols can be assessed only from a direct comparison of tests with identical specimens. For wood plywood and OSB panels such a comparison has been made in a study performed at UCSD (Gitto and Uang, 2002). A representative comparison is shown in Figures 2(a) to 2(c). The CUREE and ISO protocols produce comparable results, whereas the SPD protocol produces acceptable performance only at a clearly smaller amplitude. For all but soft soil ground motions it is claimed that the SPD protocol provides results that are too conservative.

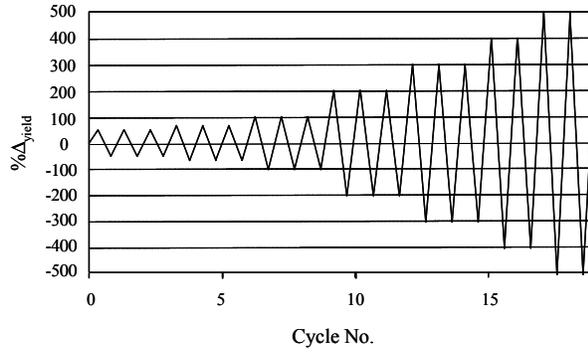
### **SHORTCOMING OF PRESENTLY EMPLOYED LOADING PROTOCOLS**

The ATC-24 and CUREE protocols are based on a statistical evaluation of inelastic time history analyses performed with SDOF systems, and using "ordinary" ground motions (distance from fault rupture greater than about 13 km to avoid near-fault effects) from earthquakes of magnitudes 7.0 or smaller. Ground motions from earthquakes greater than magnitude 7.0 are not considered (because of a lack of records). This raises the issue of long duration ground motion, which, if indeed dominant, would increase the number of large inelastic cycles. It is claimed (without proof because of lack of recorded data) that, except for ground motions at soft soil sites, the long duration issue may not dominate the loading history. The argument is that the long return period hazard in California's two major urban areas is dominated either by near-fault records with forward directivity, which are of short duration, or by large earthquakes from larger distance, which affect primarily the long period range of the spectra. There, the number of inelastic excursions is relatively small because of the few large cycles experienced by a long period structure. This statement contains conjectures that need to be justified through more research.

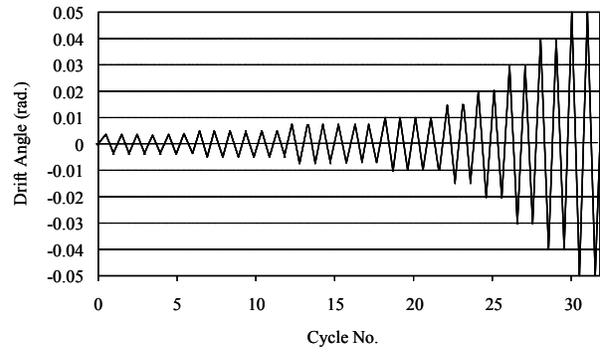
A yet unresolved issue is the duration effect of soft soil ground motions. The protocols in Figure 1(a), (b), (d), and (f) have been based on time history responses of structures subjected to site class D ground motions. As such, they are expected to be conservative (in number and relative magnitude of cycles) for site classes A, B, and C. But they likely underestimate the cumulative damage demand, represented primarily by the large cycles of the protocol, for components of structures located at sites of classes E and F. The latter two site classes deserve special consideration in the development of loading histories. The input energy of soft soil records is much larger than that of nearby rock records, and as a consequence more cycles with large deformation amplitudes has to be expected. The amplification in energy demands depends very much on the ratio of structure to soil period, and so does the amplification in the displacements response, which greatly complicates the development of a loading history that is statistically representative for structural responses to soft soil ground motions.

Most loading protocols do not contain specific requirements on the largest amplitude that should be executed in a cyclic load test. Tests are often terminated when the amplitude is large enough to satisfy specific testing objectives (such as quantifying the deformation at which the cyclic resistance has dropped to 80% of the recorded peak strength of the specimen). Terminating a test at such a relatively small decrease in resistance does not permit evaluation of important deterioration characteristics that control component response at large story drifts at which a structure approaches collapse. With the increasing emphasis on prediction of collapse capacity of structures, this is a shortcoming that in most cases is unnecessary. Modeling of deterioration, which today is based mostly on calibration against experimental data, would be much improved if all component experiments would be executed to a deformation amplitude associated with a loss in strength of at least 50%.

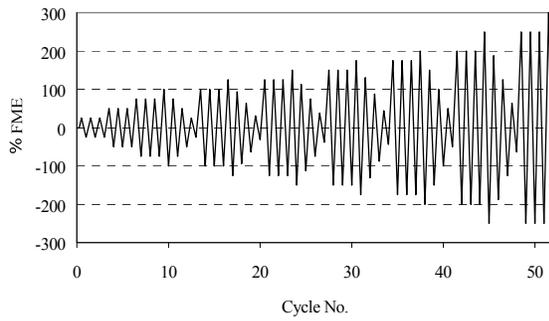
Recent analytical studies (e.g., Lignos and Krawinkler 2009) have demonstrated that representative cyclic loading histories of a typical component depend on the performance level of interest. If component behavior close to collapse is of primary interest, then in many (but not all) cases a typical component response history will be very different from the one a component experiences in a design level earthquake. In many cases the importance of cyclic deterioration diminishes because of “ratcheting” of the response of a structure, which means that the lateral deformations (story drifts) increase in one direction and load reversals become insignificant. This is illustrated in the analytical predictions shown in Fig. 3, which illustrate first story drift and first floor beam moment – rotation responses of a 20-story steel frame structure subjected to a strong ground motion that brings the structure close to collapse. Ratcheting of the response is evident in the drift response, and the consequence is that at the component level there is negligible reversal of inelastic rotations. The  $M-\theta$  response is close to that of monotonic loading test, and cyclic deterioration is negligible. This indicates the urgent need to complement conventional component tests, which are based usually on stepwise increasing symmetric loading histories derived from responses of SDOF and MDOF systems subjected to design level ground motions, with tests whose loading history pays specific attention to behavior close to collapse. In most practical cases a monotonic test fulfills this objective. The conclusion is that a comprehensive testing program for a structural component should include a monotonic test in addition to cyclic tests.



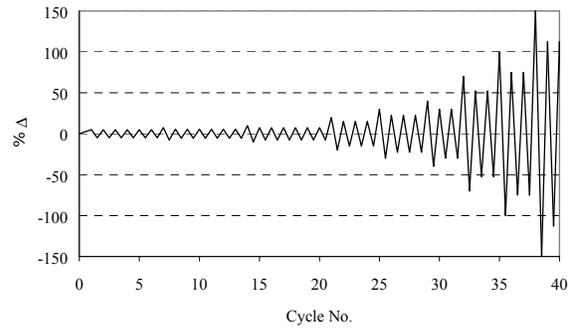
(a) Steel - ATC-24 (ATC-24, 1992)



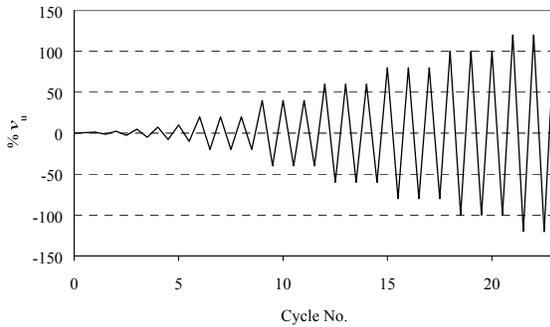
(b) Steel - SAC (Clark et al., 1997)



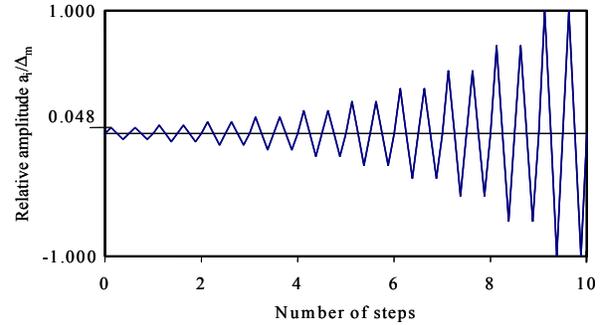
(c) Wood/Masonry - SPD (Porter, 1987)



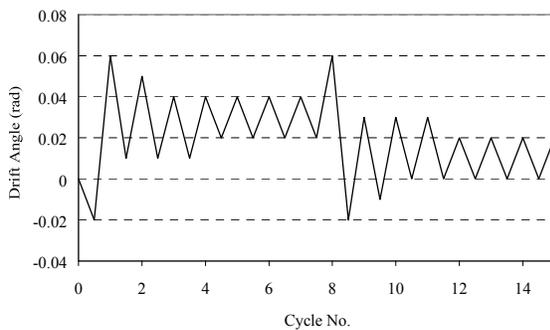
(d) Wood - CUREE (Krawinkler et al., 2000-b)



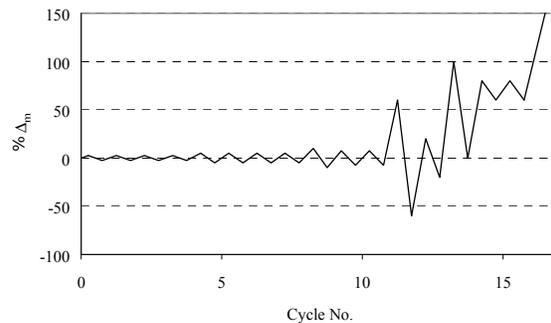
(e) Wood - ISO (ISO, 1998)



(f) FEMA 461 (FEMA, 2007)

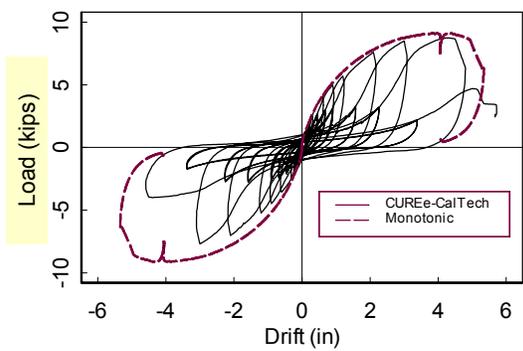


(g) Steel - SAC Near-Fault (Kraw. et al., 2000-a)

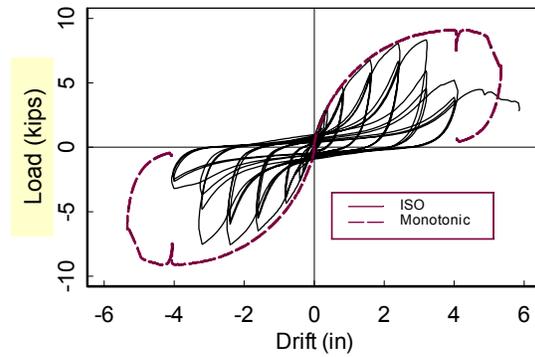


(h) Wood - CUREE Near Fault (Kraw. et al., 2000-b)

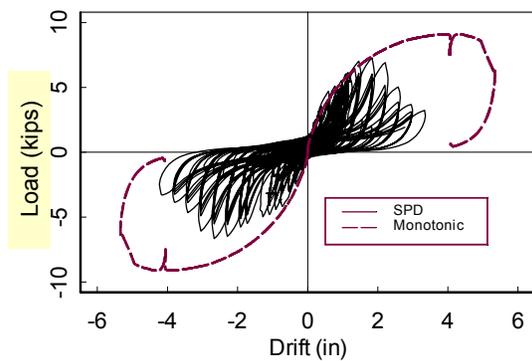
**Figure 1. Various loading protocols (several figures courtesy C.M. Uang).**



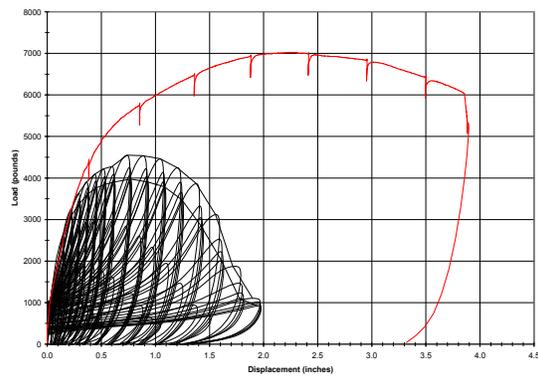
(a) OSB Wall Panel, Monotonic & CUREE Protocol



(b) OSB Wall Panel, Monotonic & ISO Protocol

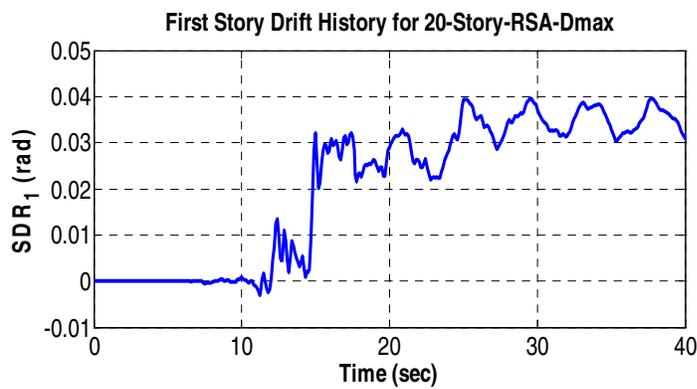


(c) OSB Wall Panel, Monotonic & SPD Protocol

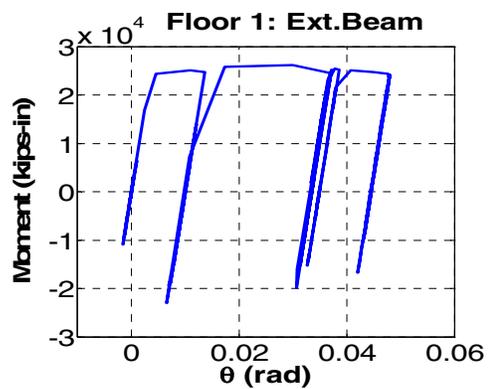


(d) Monotonic & SPD Protocol (Smith et al. 2001)

**Figure 2. Results of monotonic and cyclic test of woodframe shear wall panels. Figures (a) to (c) are for identical specimens tested at UCSD (Gitto & Uang, 2002)**



(a) First story drift response



(b) Moment-rotation response

**Figure 3. Response characteristics of a 20-story frame structure close to collapse.**

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