

Development of Loading Histories for Testing of Steel Beam-to-Column Assemblies

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Contents

Introduction	1
Proposed Loading Histories	2
1. Basic Loading History	2
2. Near-Fault Loading History	3
Commentary on Proposed Loading Histories	6
C1. General Comments	6
C2. Basic Loading History – Multiple Step Test	7
C2.1 <i>Observations and Assumptions on Capacities</i>	7
C2.2 <i>Definitions of Seismic Demand Parameters</i>	8
C2.3 <i>Considerations in Development of Basic Loading History</i>	9
C2.4 <i>Observations and Assumptions on Evaluation of Seismic Demand Parameters</i>	10
C2.5 <i>Statistics on Demand Parameters</i>	11
C2.6 <i>Observations from Data Analysis</i>	11
C2.7 <i>Approach Employed for Development of Basic Loading History</i>	12
C2.8 <i>Target Values for Demand Parameters of Basic Loading History</i>	12
C2.9 <i>Evaluation of Proposed Loading History</i>	13
C2.10 <i>Calibration Against Statistical Demands from SAC Phase II Study</i>	14
C2.11 <i>Evaluation of Basic Loading History in Relation to SAC Phase I Loading History</i>	15
C.3 Near-Fault Loading History	16
References	18
Figures	19
Appendix A	33
A1 Rationalizing the Use of Interstory Drift Angle as Basic Demand Parameter	33
A2 Deformation Control for Experimental Studies	35
A3 Deflection (Drift) Components	37
Appendix B – The Need for Cycle Counting	40
Appendix C – Summary of Statistical Information	42

Introduction

The choice of a testing program and associated loading history depends on the purpose of the experiment, type of test specimen, and type of anticipated failure mode (e.g., rapid strength deterioration such as caused by fracture, or slow strength deterioration such as caused by local buckling.).

Testing of a single specimen is adequate if the rate of strength deterioration is slow (or the level at which rapid strength deterioration occurs is well defined, e.g., global member buckling), and analytical cumulative damage modeling is not part of the investigation. Testing of a single specimen provides insufficient information if the rate of strength deterioration is rapid and the level at which deterioration occurs may exhibit considerable scatter (ATC-24, [ATC 1992]). Fracture at weldments is a failure mode that falls into the latter category. For such a case ATC-24 recommends to test at least three identical specimens with identical loading histories, and to base performance evaluation on the test with the smallest energy dissipation capacity unless a sufficient number of specimens is tested to permit a statistical evaluation of the results. Alternatively, a cumulative damage testing program (see ATC-24, Section 4.2.2) may be performed.

Two loading histories are recommended for testing of steel beam-to-column subassemblies. One is referred to as the basic loading history; it should be employed to evaluate performance of a beam-to-column subassembly provided the ground motion that controls design is not of a near-fault type that contains a large displacement pulse. In the latter case the second loading history, referred to as the near-fault loading history, should be utilized.

Proposed Loading Histories

1. Basic Loading History

The basic loading history is a multiple step test, in which the loading (deformation) history consists of stepwise increasing deformation cycles as illustrated in Fig. 1. The deformation parameter to be used to control the loading history is the interstory drift angle, θ , defined as interstory displacement over story height. In the test specimen, this angle is defined as beam deflection over beam span (to centerline of column) if the vertical beam deflection is the control parameter, or as column deflection over column height if the horizontal column deflection is the control parameter. Deformation control shall be used throughout the experiment.

In the basic loading history the cycles shall be symmetric in peak deformations (drift angle). The history is divided into steps and the peak deformation of each step j is given as θ_j , a predetermined value of the interstory drift angle. Thus, the loading history is defined by the following parameters:

- θ_j the peak drift angle in load step j
- n_j the number of cycles to be performed in load step j

Numerical values of θ_j and n_j :

Load Step #	peak drift angle θ	number of cycles, n
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2

Continue with increments in θ of 0.01, and perform two cycles at each step

Additional Considerations:

- It is recommended to interrupt the basic loading history during the last cycle of selected large steps and carry out small cycles in order to evaluate intermittent stiffness degradation. For such a stiffness evaluation the specimen is to be unloaded after reaching the peak (positive or negative) of the last cycle in the designated step, and subjected to two cycles with an amplitude of 0.005 radians, whereby the amplitude is measured with respect to the permanent deformation at the unloaded state.
- The loading history shall be continued in the established pattern until severe strength deterioration is evident. If the displacement limit of the test setup is approached before severe deterioration occurs, the test specimen shall be cycled at maximum peak deformation until severe deterioration is evident. Severe deterioration may be defined as attaining a resistance at peak deformation of less than 30% of the maximum resistance.

- If post-fracture behavior is to be investigated, the test program shall continue to the limit of the test facility, even after fracture and severe deterioration have occurred.

Acceptance Criteria:

Acceptance is based on passing pre-defined performance criteria. The maximum deformation (drift angle) amplitude at which these performance criteria are passed is referred to as the deformation capacity of the specimen. The following criteria are recommended for acceptance at the collapse prevention performance level:

- One full cycle at the maximum amplitude must be resisted by the specimen without excessive deterioration in strength.
- Excessive deterioration in strength is assumed to exist when the resistance (measured by the load applied to the test specimen) at peak drift is smaller than x% of the maximum resistance recorded during the test in the appropriate direction of loading. The value of x is to be provided by others.
- Acceptance is associated with the direction of loading. Since failure modes are direction dependent, different deformation capacities may be associated with the two directions of loading.

2. Near-Fault Loading History

This loading history is developed specifically for performance evaluation at one specific level of response, representing the effect of near-fault ground motions on SMRF behavior. Whereas the basic loading history associates acceptable performance with a deformation (story drift) capacity, the near-fault loading history provides only a pass/fail assessment for a specific direction of loading.

The near-fault loading history is one-sided to incorporate the predominant effect of a pulse contained in typical near-fault motions in large magnitude earthquakes. The history is constructed based on the response of the SAC model buildings to the SAC near-fault ground motions for the Los Angeles location.

The history of interstory drift angle θ to be applied to the test specimen is shown in Fig. 2. The first half of the history (from the beginning to point A) examines performance for one loading direction, and the second half of the history (from point A to point B) examines performance for the opposite loading direction. The second half of the history (testing in the opposite loading direction) can serve as an acceptance test only if the first half of the history has not led to observable deterioration of strength for the opposite direction of loading. If this criterion is not fulfilled, a new specimen must be used to examine acceptance in the opposite loading direction.

The loading history is displacement controlled and no acceptance judgment can be passed unless at least half of the loading history shown in Fig. 2 is completed (up to point A).

Additional Considerations:

- After completion of the acceptance test as shown in Fig. 2 (first half or both halves), loading in one direction shall be continued until severe strength deterioration is evident or the displacement limit of the test setup is attained.
- If post-fracture behavior is to be investigated, loading shall continue in one direction to the limit of the test facility, even after fracture and severe deterioration have occurred.

Acceptance Criteria:

Acceptance is based on passing pre-defined performance criteria. In this test acceptable performance is associated with a specific direction of loading and may be defined as follows.

- A complete half of the loading history must be executed, and performance is deemed acceptable if at the end of the directional loading history (at point A or point B) no excessive deterioration in strength exists.
- Excessive deterioration in strength is assumed to exist when the resistance (measured by the load applied to the test specimen) at point A or B is smaller than $y\%$ of the maximum resistance recorded during the test in the appropriate direction of loading. The value of y is to be provided by others.
- If excessive strength deterioration exists at the acceptance points, no judgment can be made on the performance of the specimen in a near-fault event.

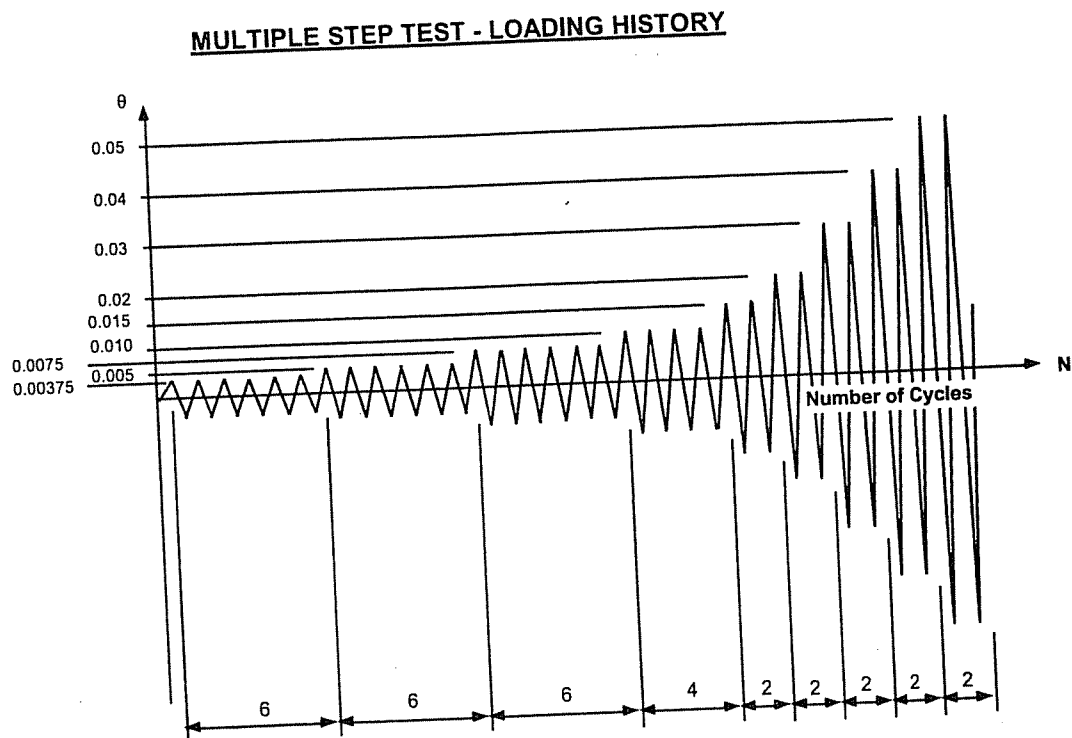


Fig. 1 Loading History for Multiple Step Test. Deformation Parameter is Interstory Drift Angle

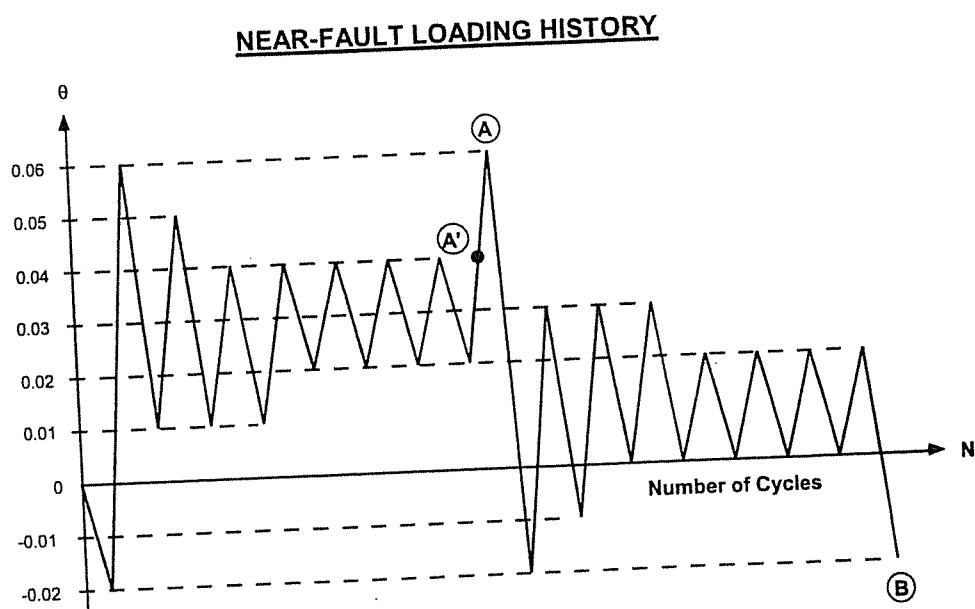


Fig. 2 Near-Fault Loading History. Deformation Parameter is Interstory Drift Angle

Commentary on Proposed Loading Histories

C1. General Comments

This discussion is concerned with loading histories for the SAC testing protocol for beam-to-column assemblies (SAC, 1997). These assemblies usually consist of one column with one or two beams framing into it. It is assumed that these assemblies constitute geometrically representative substructures of realistic frame configurations. It is further assumed that the objective is to modify, as needed, the SAC Phase I loading protocol which is based on ATC-24. There is an extensive commentary in ATC-24, which tries to rationalize the recommended loading history. This commentary is not repeated here, but the following discussion builds on this commentary. It is assumed that the reader is familiar with the nomenclature used in the ATC-24 document.

It must be emphasized that the use of a single test specimen is not recommended in ATC-24 if the specimen may experience a brittle failure mode (i.e., weld fracture). For such cases, ATC-24 recommends the use of a low-cycle fatigue testing program that requires the use of at least three identical specimens. The reason is that brittle failure modes have severe consequences (rapid deterioration) and are usually (for sure in the case of weld fracture) associated with great uncertainties in the force and/or deformation levels at which such failure modes occur. For such cases a single test will provide inconsistent information; in fact it may provide misleading information. Thus, it is not recommended to use a single specimen testing program for any "improved" connection in which a weld fracture is the likely failure mode. For such cases safety considerations have to outweigh economic considerations and a multi-specimen testing program needs to be performed. The discussion on loading histories presented here is for a single test. The issue of the number of necessary tests is not addressed here; ATC-24 provides an extensive discussion on this subject.

The demand imposed by a severe earthquake on a structural component depends on the configuration of the component within a structure, the strength and elastic and inelastic dynamic characteristics of the structure, and the seismic input to which the structure may be subjected. For generic components none of these variables is well defined or narrowly bracketed. The best mediator between component demand and seismic input appears to be the interstory drift, since this parameter can usually be related to the component deformation, and the demand on this parameter can be assessed from simplified dynamic models. Thus, the basic deformation demand parameter is assumed to be the interstory drift angle θ . This is the control parameter to be used for loading history control.

For steel moment frame structures the story drift angle is an evident choice for the control parameter. It is a parameter whose value in a given earthquake does not appear to be very sensitive to the number of stories and to subjective design decisions. Extensive analytical studies (Gupta and Krawinkler, 1999) have shown that the story drift (in terms of amplitude and other measures of interest for cumulative damage evaluation) can be bracketed except in extreme cases. Moreover, also the yield value can be bracketed relatively well. For this reason the decision was made here to use absolute values of drift angle as the measure upon which to structure the individual cycles of the loading history. This choice is not the only option. For instance, in a recently developed testing protocol for wood structures, the maximum deformation that can be

sustained by a component according to a predetermined performance criterion is used as the quantity upon which the individual cycles of the loading history are structured (Krawinkler et al., 2000). There are pros and cons in both options, but for steel frame structures the first option is clearly the better one.

Additional reasons why θ is selected as the basic demand parameter, and relationships between θ and local demand parameters (rotations in beams and columns, and shear distortion in joint panel zone) are summarized in Appendix A.

C2. Basic Loading History – Multiple Step Test

In the development of this loading history it is assumed that ground motions cause several cycles of damaging response and are not of a near-fault type that contains a large displacement pulse. A loading history for the latter cases is discussed in Section C3 of this Commentary.

C2.1 Observations and Assumptions on Capacities

All seismic capacity parameters (strength, stiffness, inelastic deformation capacity, and cumulative capacity parameters such as energy dissipation capacity) are expected to deteriorate as the number and amplitude of cycles increase. It is assumed that the onset of deterioration, as well as the rate of deterioration, can be described by a cumulative damage model of the type (Krawinkler et al., 1983)

$$D = C \sum_{i=1}^N (\Delta\delta_i)^c \quad (C.1)$$

where

- $\Delta\delta_i$ = deformation range (total change in deformation) of excursion (or cycle) i
- N = number of damaging excursions (or cycles)
- C = a structural performance parameter that may depend strongly on the type of component and failure mode
- c = a structural performance parameter the is usually greater than 1.0.

The type of deterioration depends on the failure mode of the component. Figure C.1 illustrates examples of deterioration for two distinctly different failure modes; slow and gradual deterioration (e.g., local buckling of beam flanges), Fig. C.1(a), and rapid deterioration (e.g., weld fracture), Fig. C.1(b).

Each mode of deterioration and failure has its own characteristics that may affect the choice of testing program and loading history. Elements with crack propagation and fracture at weldments exhibit usually rapid strength deterioration of the type illustrated in Fig. C.1(b). The level of force (or deformation) at which this rapid deterioration (unstable crack growth) occurs may have considerable scatter. In this case, a single test provides only one data point which cannot be used with confidence for a quantitative evaluation of performance. If slow deterioration occurs, the

scatter of results is expected to be smaller, the consequences of the scatter are less severe, and the results of a single test are likely adequate to provide the information needed to assess the capacity (in terms of force, deformation, or energy dissipation) of the element.

Generic loading histories, which are to be applied to different specimens with different failure modes, represent a compromise that is based on important performance characteristics that are common to all specimens tested. The recommended loading histories are based on a general cumulative damage concept of the following characteristics:

- Every excursion causes damage in an element. Damage implies that macro- or microstructural changes occur, which cause visible or invisible deterioration of strength and stiffness properties and bring the component closer to failure.
- The component has a memory, i.e., the damage from inelastic excursions is cumulative.
- Large excursions cause much larger damage than small excursions ($c = 1.0$).
- In general, the relative amount of damage caused by an excursion depends on the deformation range of the excursion (or cycle), $\Delta\delta$, the mean deformation of the excursion (a measure of symmetry with respect to the undeformed configuration), and the sequence in which large and small excursions are applied to the component (sequence effects).
- For a given deformation amplitude the damage is largest for a symmetric excursion (or cycle) since this results in the largest possible deformation range.
- The importance of sequence and mean effects on the behavior of steel components has not yet been established through research, and the sequence of large vs. small excursions in a component of a structure subjected to a severe earthquake does not follow a consistent pattern. Thus, the basic loading history is one in which sequence and mean effects are not being considered.
- As a consequence, for the basic loading history the number of cycles (N), the deformation ranges of the cycles ($\Delta\delta_i$), and the sum of the deformation ranges ($\Sigma\Delta\delta_i$) become the primary capacity parameters, and the history should consist of cycles of stepwise increasing deformation amplitude.
- The importance of sequence and mean effects needs to be investigated through a special testing program that focuses on the response to near-fault ground motions, characterized by a large excursion (or cycle) followed by a series of smaller cycles.

C2.2 Definitions of Seismic Demand Parameters

From here on the interstory drift angle θ will be used as the basic deformation parameter, which in the previous section was denoted as δ . With this definition the demand parameters on which the basic loading history is based are the following:

N_t = total number of cycles with deformation (interstory drift angle) range = 0.005.
[This range is selected because a deformation amplitude of 0.0025 (half of the range) corresponds to a conservative estimate of half the yield deformation.]

θ_{max} = maximum deformation (interstory drift angle) experienced during the seismic response. [This is an amplitude value, used only to judge symmetry, and considered because of its perceived importance.]

$\Delta\theta_{max}$ = maximum deformation range experienced during the seismic response.

$\Delta\theta_2$ = second largest deformation range experienced during the seismic response.

$\Delta\theta_3$ = third largest deformation range experienced during the seismic response.

$\Delta\theta_i$ = deformation range of cycle i experienced during the seismic response. [Individual cycles are not observed in the time history response. They are obtained by performing rainflow cycle counting on the interstory drift angle response time history. See Appendix B.]

$\Sigma\Delta\theta_i$ = sum of the deformation ranges = 0.05 (after rainflow cycle counting).

$\Delta\theta_{pi}$ = plastic deformation range of cycle i . [The plastic deformation range is equal to $\Delta\theta_i - 2\theta_y$.]

$\Sigma\Delta\theta_{pi}$ = sum of the plastic deformation ranges (after rainflow cycle counting).

N_p = number of inelastic cycles (cycles with amplitude $> \theta_y$).

C2.3 Considerations in Development of Basic Loading History

A loading history that covers all bases (ground motion intensity and frequency content, and structural periods, configurations, and failure modes) does not exist. The following considerations enter in rationalizing a loading history that can be utilized for a general performance evaluation at various performance levels and in different seismic regions:

- The loading history should represent a "reasonable and generally conservative" demand on N_t , $\Delta\theta_i$, and $\Sigma\Delta\theta_i$ for the full range of anticipated interstory drift ratios (i.e., for structures of all periods, all stories in a structure, all reasonable designs, all seismic regions, all types of ground motions, etc.).

- "Reasonable and generally conservative" implies that the total number of damaging cycles, N_i , should be represented in average, and that the cumulative deformation range, $\Sigma\Delta\theta_i$, should be represented conservatively. Consideration should also be given to the fact that small cycles are much more frequent than large ones, and that small elastic cycles contribute very little to damage [unless very early brittle fracture occurs at a connection].
- Primary consideration should be given to the cycles with relatively large deformation ranges, which will dominate damage accumulation.
- Additional consideration should be given to a conservative representation of the plastic deformation ranges.
- Even though desirable (see first bullet), it will not be possible to separate the loading history fully from the maximum deformation range, $\Delta\theta_{max}$, at which acceptability is to be evaluated. This cannot be done because $\Sigma\Delta\theta_i$ depends strongly on $\Delta\theta_{max}$, and $\Sigma\Delta\theta_i$ is the most important parameter to be represented in the loading history.

C2.4 Observations and Assumptions on Evaluation of Seismic Demand Parameters

- The SAC model buildings (3, 9, and 20 stories) designed for Los Angeles and Seattle form the basis for demand evaluation.
- The SAC ground motion records (10/50 and 2/50) at the three locations are employed for demand evaluation.
- The original development of the demand data on which the basic loading history is based was done with ground motion records that are somewhat different from the records used in the SAC Phase II study (different scaling, and components rotated by 45°). Even though the quantitative demand evaluation is ground motion dependent, it has been confirmed that the use of updated SAC records and statistical definitions would not have changed the loading history decisions made in the early development. Thus, the process and data used in the early development are documented here. A calibration against data obtained from the final SAC Phase II study is presented later in this section.
- In the basic loading history development, emphasis is placed on the LA buildings and ground motion records (10/50 and 2/50 sets). The response data presented in this section are based on dynamic analysis of the LA model buildings subjected to fault normal and fault parallel components of ground motion records.
- Predictions of interstory drift angle demands are based on 2-D analyses, using the simplest analytical model (centerline dimensions; no joint distortions are considered, i.e., model M1 [Gupta and Krawinkler, 1999]). [In the calibration against data from the final SAC Phase II study the improved model M2 is used.]

- The interstory drift response time history varies greatly between records. It has a pattern that is either reasonably symmetric with respect to the undeformed configuration, or exhibits pulse type characteristics representative of the response to near-field ground motion (see Fig. C.2). It must be emphasized that the LA 2/50 set of records consists mostly of near-fault records.
- Damaging excursions (or cycles) are identified by applying a cycle counting method to the deformation histories obtained from analysis. In order to arrive at closed cycles (every excursion in one direction has a counterpart in the opposite direction), the simplified rainflow cycle counting method is applied (see Appendix B).
- Thus, the starting point for the loading history development is a series of ordered (in magnitude) deformation (interstory drift angle) ranges, $\Delta\theta_i$, for each story in each structure, and for each ground motion record. All parameters identified in the previous section are derived from this set of data.

Figure C.3 shows typical values of response parameters for the 3-story LA structure (40 records: 1 to 20 = 10/50 records, 21 to 40 = 2/50 records). The large demands observed in the responses to several of the 2/50 records come from fault-normal components of near-fault records. [After this analysis effort SAC decided to rotate the fault normal and fault parallel components of the records by 45° for subsequent analysis studies.]

C2.5 Statistics on Demand Parameters

Statistical evaluation of demand parameters is performed for the "critical" stories of each structure. Critical implies largest values in θ_{max} , $\Delta\theta_{max}$, and $\Sigma\Delta\theta_i$. [For the 3-story and 9-story LA structures the 3rd story and 9th story, respectively, are found to be critical.]

- For each set of data (e.g., $\Delta\theta_{max}$ for the third story of the LA structure, using the 10/50 record set) several probabilistic distributions have been tested. The distribution that shows the best visual fit is selected.
- For each set of data the 50, 75, and 90 percentile values are computed.
- Based on past studies (Krawinkler et al., 1983) it is assumed that the individual deformation ranges for each record follow a lognormal distribution. Thus, order statistics can be employed and the data from all records can be used to derive a lognormal distribution of the deformation ranges. Specific values of the lognormal distribution parameters are not derived here, but the CDF graph of all the data points is used as the basis for determining the amplitudes of the individual cycles of the basic loading history (discussed later).

C2.6 Observations from Data Analysis

- The parameter variations for the 3rd story of the LA 3-story building and the 9th story of the LA 9-story building are as shown in Fig. C.4. In these stories the demands are highest for the 3- and 9-story buildings. Results are presented for the 50, 75, and 90 percentiles, and for three record sets (10/50-20 records, 2/50-20 records, and 2/50-10 records [the latter referring to the

10 fault normal components of the 2/50 records]). Results are presented for N_i , θ_{max} , $\Delta\theta_{max}$, $\Delta\theta_2/\Delta\theta_{max}$, $\Sigma\Delta\theta_i$, and $\Sigma\Delta\theta_{pi}$.

- The 3rd story of the 3-story LA structure exhibits the highest demands.
- N_i appears to be the only parameter not very sensitive to the record set.
- No separate statistics was done on the ratios $\Delta\theta_2/\Delta\theta_{max}$ and $\Delta\theta_3/\Delta\theta_{max}$. The plotted values are the ratios of the statistical values of the components, i.e., $(\Delta\theta_2/\Delta\theta_{max})_{75} = (\Delta\theta_2)_{75}/(\Delta\theta_{max})_{75}$. The data show that these ratios are relatively small, and are only weakly dependent on the record set.
- All other parameters exhibit significant sensitivity to the record set and indicate large scatter of the data. This holds true particularly for the 2/50 record sets.

C2.7 Approach Employed for Development of Basic Loading History

The basic loading history consists of a series of steps with increasing deformation (interstory drift angle) range, $\Delta\theta_i$. At each step a predetermined number of cycles is to be performed. The number of cycles and the deformation range in each step are based on the following considerations.

- Only symmetric cycles are to be performed, i.e., the deformation amplitude is equal to half the deformation range, $\Delta\theta_i$.
- The number of cycles and the magnitude of the interstory drift angle range in each step are determined in a manner such that for a selected number of cycles, N_i ,
 - the CDF of the test loading history is below the CDF of the $\Delta\theta_i$ data points determined by analysis for the 10/50 and 2/50 sets of ground motion records, and
 - the sum of the deformation ranges of the loading history is larger than the demand on $\Sigma\Delta\theta_i$ for the 10/50 and 2/50 record sets at the appropriate statistical level (percentile).
- Appropriate choices need to be made for the statistical levels (percentiles) at which the demands for N_i , $\Delta\theta_{max}$, and $\Sigma\Delta\theta_i$ are to be met or exceeded in the loading history.

C2.8 Target Values for Demand Parameters of Basic Loading History

The third story of the LA 3-story building exhibits the largest demands. Thus, the analysis results for this interstory drift angle are used to set the target values and develop a loading history. The adequacy of the loading history is then checked against the computed demands for the 9th story of the LA 9-story building. Statistical values for the demands are documented in Appendix C and plotted in Fig. C.4.

Number of Damaging Cycles, N_t . It is assumed that every cycle with $\Delta\theta > 0.005$ causes damage and its effect should be represented in the loading history. This is the basis for the computed N_t values. As Fig. 4 shows, N_t is not very sensitive to the record set. Since N_t should be represented in average, a value of approximately 30 is reasonable. Thus $N_t = 30$ is used as a target.

Maximum Deformation Range, $\Delta\theta_{max}$. For the 10/50 record set the probability of exceedence of the target value should be very low. Thus, the 90 percentile value, i.e., $\Delta\theta_{max} = 0.0504$, is chosen to set a target. The closest range value of the loading history is 0.06 (range of 0.06 implies amplitude of 0.03).

For the 2/50 record sets a higher probability of exceedence should be acceptable. The 75 percentile is chosen for the set of 20 records, i.e., $\Delta\theta_{max} = 0.0839$. For the set of 10 fault normal components the 50 percentile value is chosen, i.e., $\Delta\theta_{max} = 0.0839$. The closest range value of the loading history is 0.08.

Maximum Deformation θ_{max} . For the same probabilities of exceedence the following statistical values are obtained:

10/50 set:	$\theta_{max} = 0.0332$
2/50 set:	$\theta_{max} = 0.0522$
2/50-10 set:	$\theta_{max} = 0.0534$

The latter two values exceed the recommended maximum test amplitude of 0.04. This is not considered a problem for two reasons. First, damage is proportional to range and not amplitude, and it is very unlikely that an equally severe excursion occurs in the opposite direction. Second, the largest response values for θ_{max} are obtained from near-field type ground motion records which pose special problems.

Cumulative Deformation Range, $\Sigma\Delta\theta_i$. The same probabilities of exceedence should apply as for the maximum deformation range. The corresponding statistical values are:

10/50 set:	$\Sigma\Delta\theta_i = 0.470$
2/50 set:	$\Sigma\Delta\theta_i = 0.487$ (for 90 percentile: $\Sigma\Delta\theta_i = 0.602$)
2/50-10 set:	$\Sigma\Delta\theta_i = 0.420$ (for 90 percentile: $\Sigma\Delta\theta_i = 0.702$)

C2.9 Evaluation of Proposed Loading History

The proposed loading history was developed with the above targets in mind, and paying attention to the following additional considerations:

- The number of small cycles is much greater than the number of large cycles.
- The step increment should be relatively small at small ranges, but can increase at large ranges.
- The steps should be such that the second and third largest deformation ranges are reproduced conservatively.

The matching of the loading history with the statistical targets is evaluated in Fig. C.5 for the following four cases:

- 3rd story of 3 story LA building, 10/50 record set
- 3rd story of 3 story LA building, 2/50 record set
- 9th story of 9 story LA building, 10/50 record set
- 9th story of 9 story LA building, 2/50 record set

Indicated on each graph are

- The number of cycles at each step of the loading history
- The statistical values of the largest (1), second largest (2), and third largest (3) range
- The cumulative deformation range from the test and the statistical target.

C2.10 Calibration Against Statistical Demand Data from SAC Phase II Study

The objective of the basic loading history is to subject a test specimen to a cumulative deformation demand that exceeds, for any postulated maximum deformation demand, the cumulative demand the component represented by the specimen may experience in an earthquake associated with a specific hazard level. Thus, representative data for maximum deformation demands, θ_{max} , and associated cumulative deformation demands ($\Sigma\Delta\theta_i$ and $\Sigma\Delta\theta_{pi}$) need to be obtained to evaluate how well the loading history represents demands imposed by earthquakes. The comprehensive demand assessment performed in Phase II for the SAC model buildings provides this information (Gupta and Krawinkler, 1999).

Figures C6(a) to (d) present statistical data from the SAC Phase II study and their fit to the basic loading history, which is represented by the stepped diagram. The x-marks identify the points at which acceptance is evaluated for specific maximum deformation demands (after the first complete cycle at the maximum amplitude). The marked data points represent pairs of statistical values (median and 84th percentile as defined in Gupta and Krawinkler, 1999, of maximum deformation (story drift) and cumulative deformation demands. Even though the coordinates of the data points are not correlated (e.g., the 84th percentile of the cumulative drift demand does not correspond to the 84th percentile of the maximum drift demand), the data points provide a picture of the goodness of fit of the loading history to statistical values of demand predictions.

The data points of Figures C6(a) and (b) are obtained for critical stories (maximum demands) of the LA and Seattle pre-Northridge structures subjected to the 2/50 ground motion records. The following 7 cases are shown

- | | |
|------------------------------------|-----------|
| 3rd story of LA 3-story structure | (LA-3/3) |
| 2nd story of LA 9-story structure | (LA-9/2) |
| 9th story of LA 9-story structure | (LA-9/9) |
| 3rd story of LA 20-story structure | (LA-20/3) |
| 3rd story of SE 3-story structure | (SE-3/3) |
| 2nd story of SE 9-story structure | (SE-9/2) |
| 9th story of SE 9-story structure | (SE-9/9) |

The two figures show that maximum and cumulative demands vary greatly between structures, stories, and geographic regions. This confirms that no one history can represent a good fit to all cases and demonstrates the need for a conservative history so that the demand data points fall in general below the demands simulated in the experiment.

Figure C6(a) shows that this conservatism is achieved for all of the LA cases but not for some of the Seattle cases - if the sum of deformation ranges ($\Sigma\Delta\theta_i$) is used as the cumulative damage parameter. The reason for the very large cumulative values for the Seattle structures is the very long strong motion duration of several records in the Seattle 2/50 record set. These records cause very large $\Sigma\Delta\theta_i$ values, but most of this cumulative demand is caused by a large number of elastic cycles. If only plastic deformation ranges ($\Sigma\Delta\theta_{pi}$) are considered, the cumulative demands computed for the Seattle structures are safely below those provided by the loading history (see Fig. C6(b), which shows $\Sigma\Delta\theta_{pi}$ values for the cases of Fig. C6(a) [the $\Sigma\Delta\theta_{pi}$ for the illustrated loading history is based on bounding assumptions of $\theta_y = 0.005$ and 0.01]). In fact, in all cases the computed $\Sigma\Delta\theta_{pi}$ is much smaller than provided by the loading history, which indicates that the loading history is very severe since the majority of damage is usually caused by inelastic cycles.

Figures C6(c) and (d) present results for the LA-3/3 and SE-3/3 cases, but using the 10/50 record sets. The assessment is similar to that for the 2/50 record sets.

C2.11 Evaluation of Basic Loading History in Relation to SAC Phase I Loading History

Since the SAC Phase I loading history is based on plastic deformations, it is necessary to make an assumption on the relation between specimen yield displacement and interstory drift angle θ . In the Phase I testing programs the "yield interstory drift" (beam tip displacement at yielding over beam length [to column centerline]) was usually smaller than 0.01. Thus, assuming $\theta_y = \delta_y/L = 0.01$ is close to an upper bound of the reference yield deformation, and using $\theta_y = \delta_y/L = 0.005$ is likely a lower bound of the reference yield "deformation". For the SAC test structures the computed yield drift is also between 0.005 and 0.01.

The cumulative deformation ranges ($\Sigma\Delta\theta_i$) of the Phase 1 and Phase 2 loading histories are compared in Fig. C.7, using $\theta_y = 0.01$ and $\theta_y = 0.005$ for the Phase 1 history. The following observations are made:

- If $\theta_y = 0.005$ is used for the Phase I history, then the cumulative deformation range of the Phase I history exceeds that of the Phase 2 history at $\Delta\theta = 0.07$. This range corresponds to an amplitude of $\theta = 0.035$. Subtracting the yield rotation of 0.005, the plastic rotation amplitude is 0.03, which is the acceptance value used in Phase I. Thus, at this acceptance value the Phase I history is about equally severe as the Phase II history, for $\theta_y = 0.005$.
- Using $\theta_y = 0.01$ for the Phase I history will result in cumulative deformation ranges that will always remain below those of the Phase II history. The main reason is that in the Phase II history many more small cycles are performed because the decision was made that all cycles with a range of 0.005 causes damage. The first 18 cycles are at or below yield if $\theta_y = 0.01$. With few exceptions (brittle connections that fracture very early) it is unlikely that these small

cycles contribute much to cumulative damage. Disregarding these small cycles, the cumulative deformations between the Phase I and Phase II histories are comparable.

C.3 Near-Fault Loading History

Near-fault ground motions deserve special considerations. The response to them is often characterized by one large excursion, followed by a number of small cycles with a large mean deformation. It is expected that the cumulative damage is dominated by the large excursion, which corresponds to monotonic loading of the test specimen. However, the subsequent smaller cycles may lead to additional deterioration that needs to be evaluated.

No specific attention has been paid to the characteristics of near-fault response in the development of the basic loading history. If near-fault effects dominate the response, acceptance of a component should be based on a loading history that considers these effects. Near-fault response implies fewer large excursions but the existence of one very large excursion that exceeds the maximum amplitude considered for acceptance in the multiple step test. Also, mean effects of the smaller excursions following the large one need to be considered.

The time history response data obtained from subjecting the SAC LA model structures to the fault-normal component of the SAC LA near-fault ground motion records are used to derive response characteristics that need to be considered in the near-fault loading history. The following observations made from the response evaluation affected the development of the loading history:

- Many but not all of the interstory drift response time histories show the characteristics outlined in the previous paragraphs. Two typical response histories are shown in Fig. C8. In general, the history is dominated by a relatively large excursion in one direction followed by a very large excursion in the opposite direction, followed by several smaller cycles with a large mean drift.
- Because of the one-sided nature of the response, the basic rainflow cycle counting method had to be employed rather than the simplified method (see Appendix B).
- The maximum demands (largest excursion and cumulative plastic deformations) occur in the third story of the LA 3-story structure. Statistical values for this story and for specific stories of the LA 9- and 20-story structures are presented in Fig. C9. Thus, the near-fault loading history is modeled after the response of the 3rd story of the LA 3-story structure.
- For this story the data presented in Fig. C9(a) and C10 are considered. Figure C10 shows statistical values of important excursion ranges ($\Delta\theta_i$) in the pulse direction, ordered in decreasing magnitude.

Based on the presented data and a study of the drift time history responses the near-fault loading history shown in Fig. 2 is developed. The following arguments are used in the decision process on the specifics of the history.

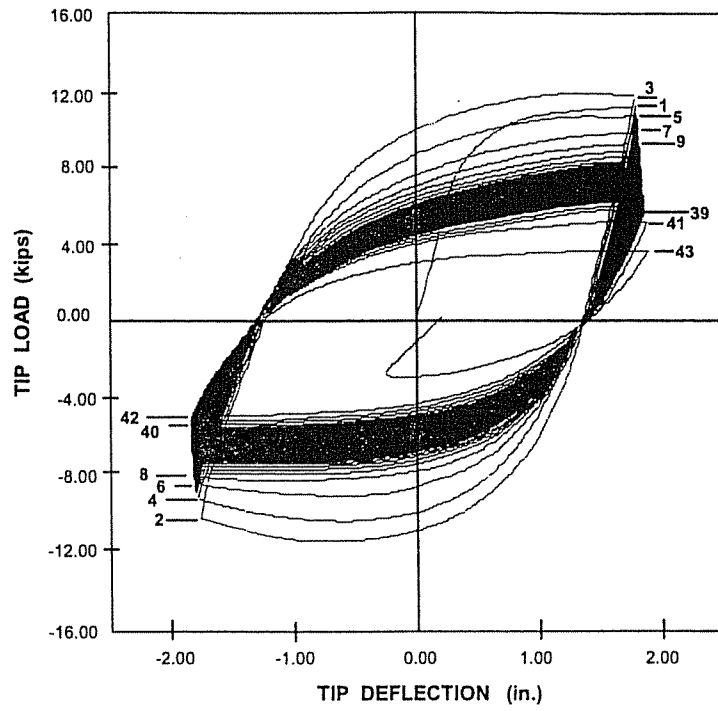
- Because the near-fault ground motions represent relatively extreme events, the median rather than 84th percentile values are used to decide on amplitudes and ranges.

- The maximum drift range is about 0.08 and the maximum amplitude is about 0.06. Thus, the first two excursions are set to go from 0 to -0.02 to +0.06.
- The next larger ranges (the first going in the negative direction and the next in the positive (pulse) direction) are approximately 0.05 and 0.04.
- From then on attention is being paid primarily to simulating the excursions in the positive (pulse) direction and ending up with a residual drift of 0.03 (the computed median residual drift is 0.0296). Statistical data for these excursions are presented in Fig. C10. The values selected for the loading history are hand-written next to the computed medians.
- It is deemed sufficient to replicate the eight largest excursions in the pulse direction, since the following smaller excursions are expected to cause negligible damage compared to the previously executed excursions.
- An acceptance evaluation at the end of the simulated history (point A') is ambiguous because adequate performance needs to be based on acceptable strength deterioration, and this deterioration may not be measurable at the peak of the last excursion. Thus, the last excursion is continued to the peak drift amplitude of 0.06 (point A) to provide an unambiguous measure of deterioration.
- The history from the origin to point A is applied to check acceptance in the positive loading direction. To check acceptance in the negative direction, the same history needs to be applied, in concept, in the opposite loading direction. In general, this requires a new specimen because it is expected that loading in one direction will have an effect on the resistance in the opposite direction. Based on the following arguments it may be possible to utilize the same specimen for checking acceptability in the opposite direction by appending a "negative direction loading history" (from point A to point B) after completion of the "positive direction loading history".
 - The loading applied to check acceptance in the positive direction will not improve behavior in the negative direction; it already will cause damage in the negative direction because of the presence of excursions in the negative direction. Thus, the initial condition for the appended "negative direction loading history" are worse than for a new specimen.
- One could argue to extend the first excursion from point A to a negative drift angle amplitude of -0.06, to mirror the history in the positive direction. However, then the first excursion has a drift range of 0.12, which is excessively large. On the other hand, it can be argued that mean effects are relatively small and their importance is exceeded by the damage done in the first part of the loading history. If this argument is accepted, then only the drift range of the first large excursion of the reference response data needs to be simulated. This drift range is 0.08, which brings the drift angle a final value of -0.02.
- From there on the loading history executed for the positive direction will be repeated, but with the sign of all excursions reversed. Point B becomes the evaluation point for acceptance in the negative direction.

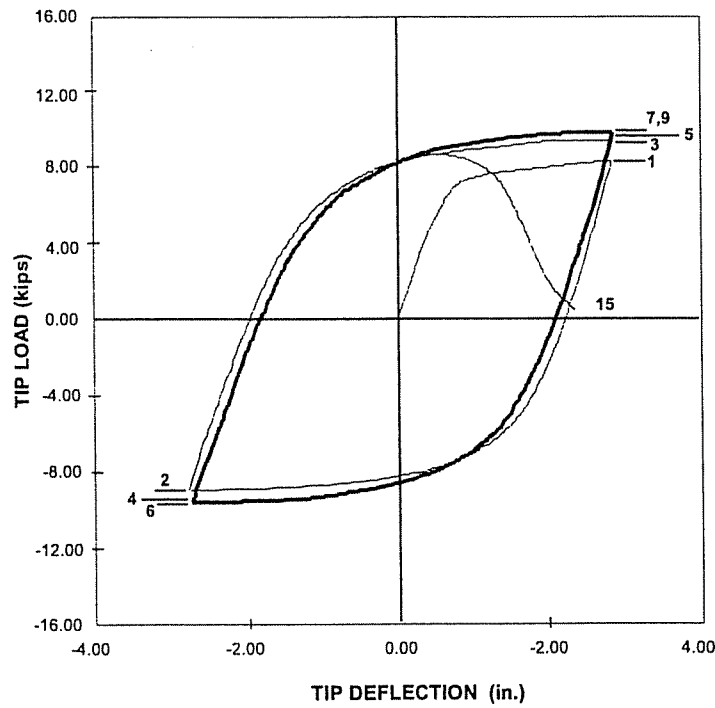
- There is one important caveat to these arguments for using the same specimen to test for acceptance in both directions. The loading history from the origin to point A may have triggered deterioration in strength in the negative direction. If this happens, the reference value for checking acceptance (maximum resistance recorded during the test in the appropriate direction of loading) will be affected. Moreover, any such deterioration in strength may affect the final failure mode and may distort the component assessment. This holds true particularly if weld fracture is a possible failure mode. A deterioration in strength, which may have been caused by local buckling during the "positive direction loading history", will decrease the level of stress at the welded joint and, therefore, will distort the acceptance criterion. Thus the same specimen can be used for testing in the negative direction only if no appreciable deterioration of negative strength has occurred during application of the positive direction loading history.

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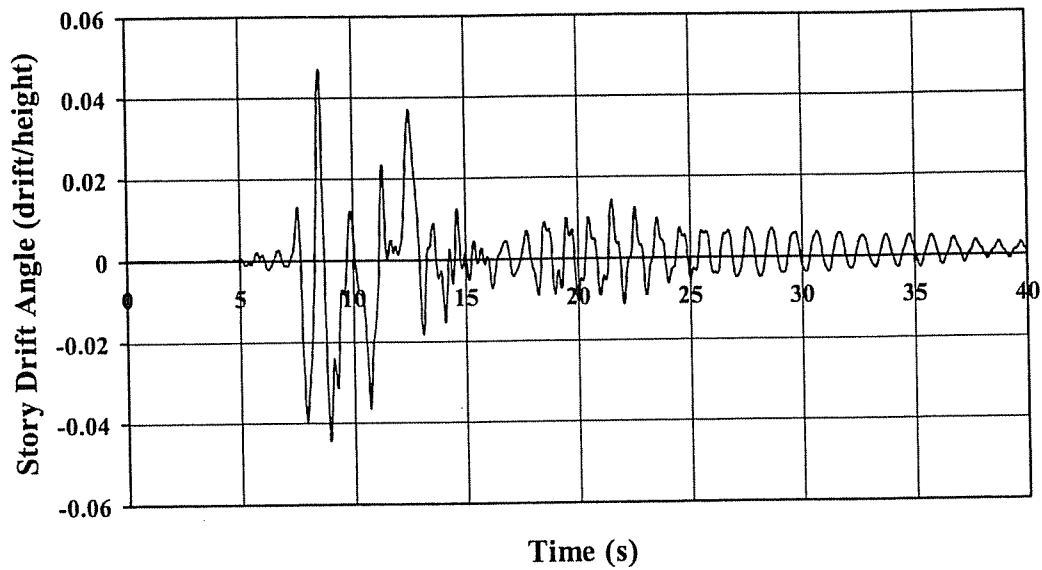
(a) Slow Deterioration



(b) Rapid Deterioration

Fig. C.1 Examples of Gradual and Rapid Deterioration

TIME HISTORY OF STORY DRIFT ANGLE : STORY 3
LA 3 Story Pre-Northridge Design, EQ: LA21, Model 1



TIME HISTORY OF STORY DRIFT ANGLE : STORY 3
LA 3 Story Pre-Northridge Design, EQ: LA37, Model 1

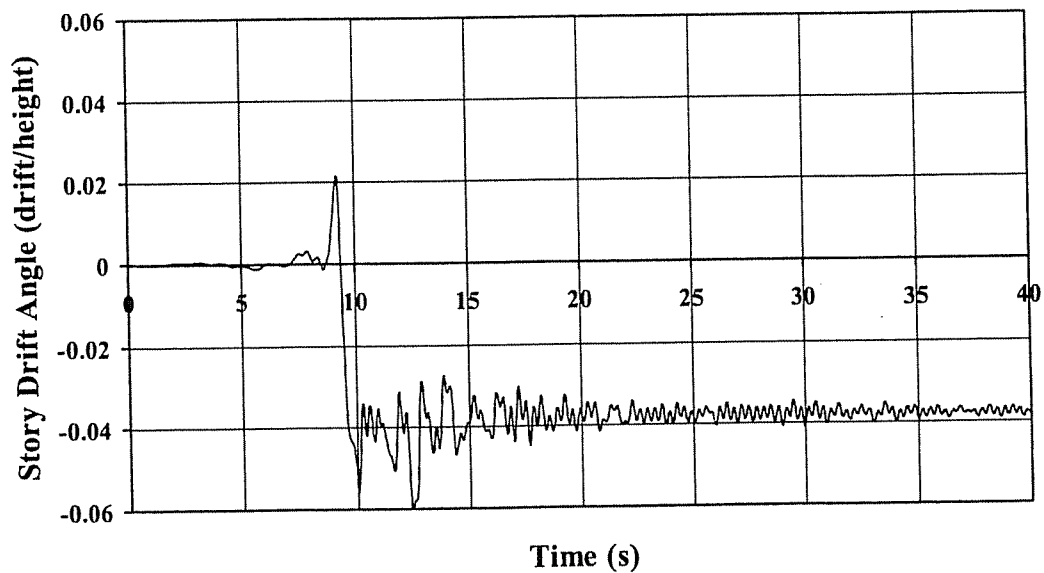


Fig. C.2 Examples of Response Time Histories (Interstory Drift Angle, Story 3, 3-Story LA)

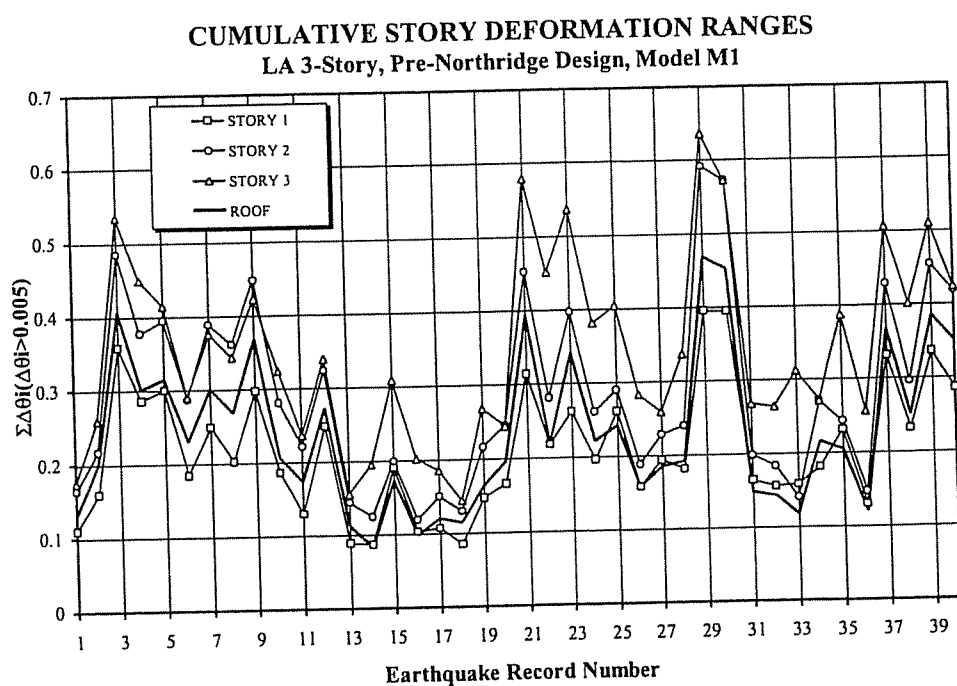
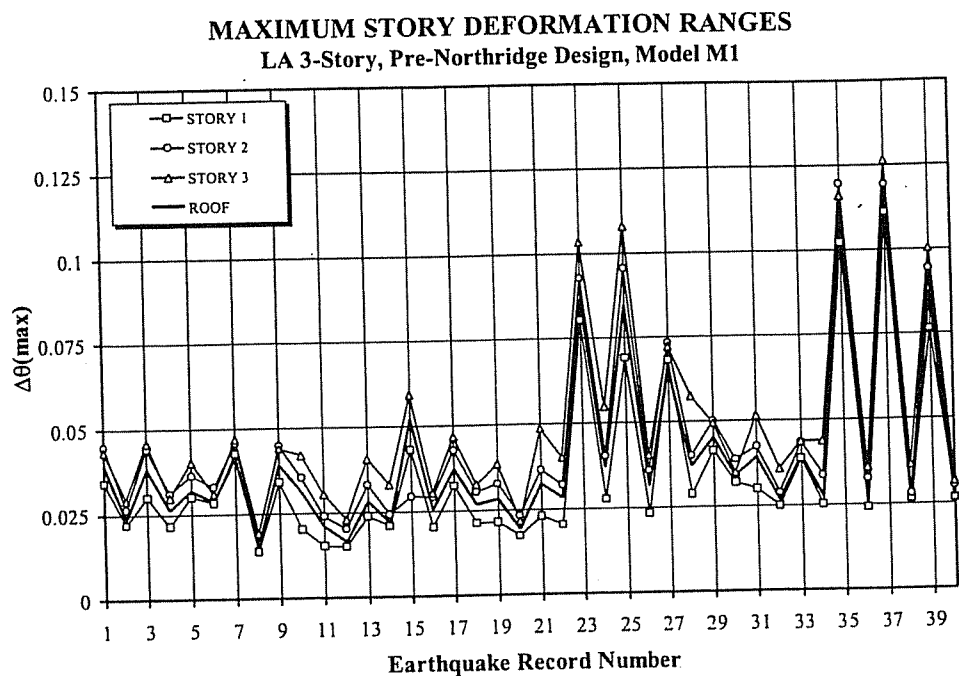


Fig. C.3 Peak Response Parameter Values

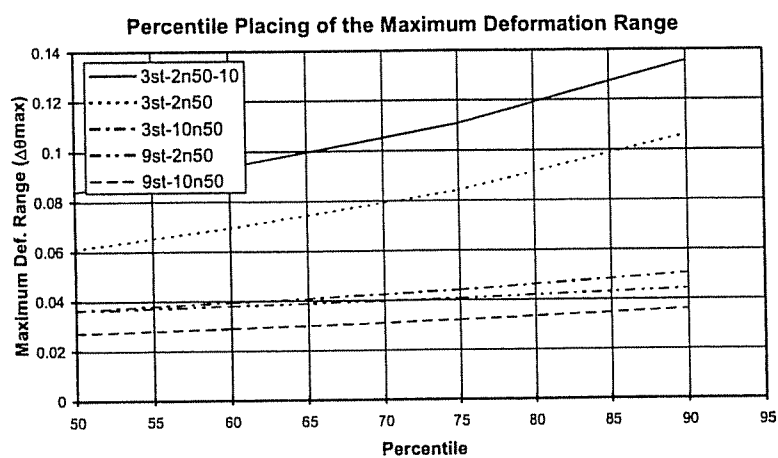
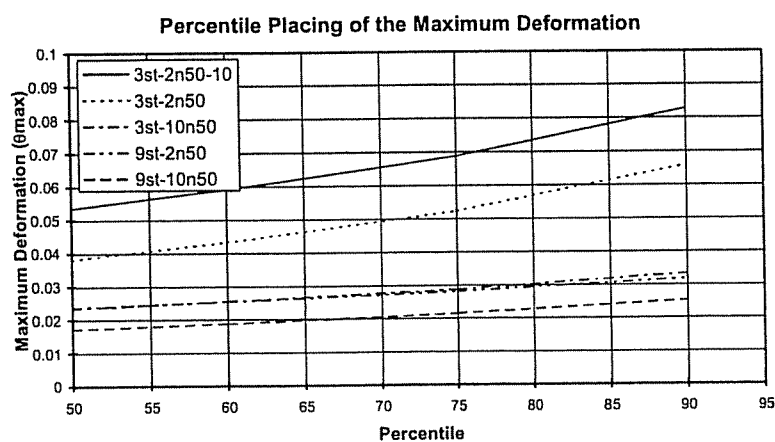
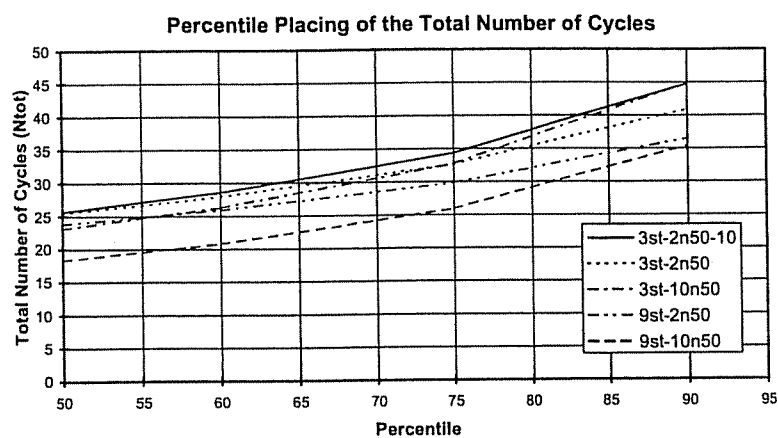


Fig. C.4(a) Statistical Values of Response Parameters

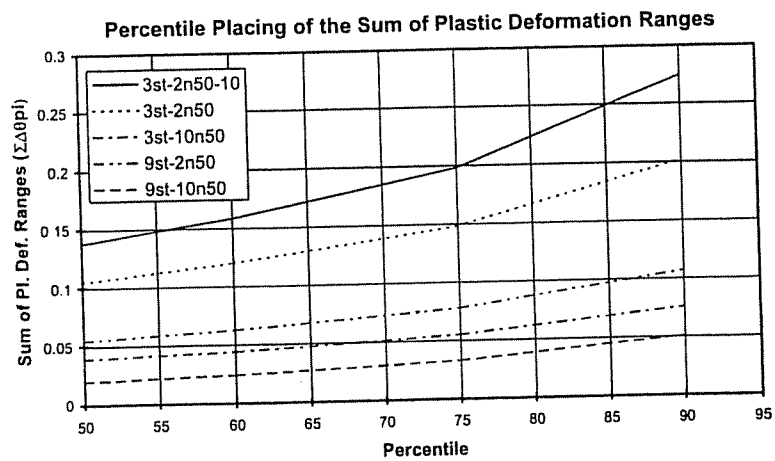
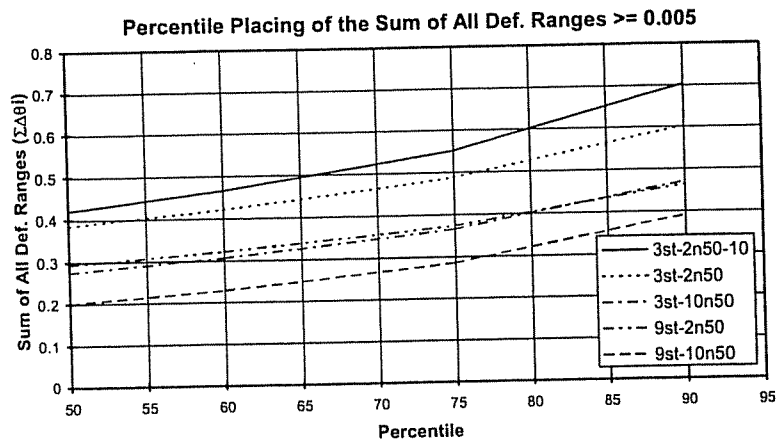
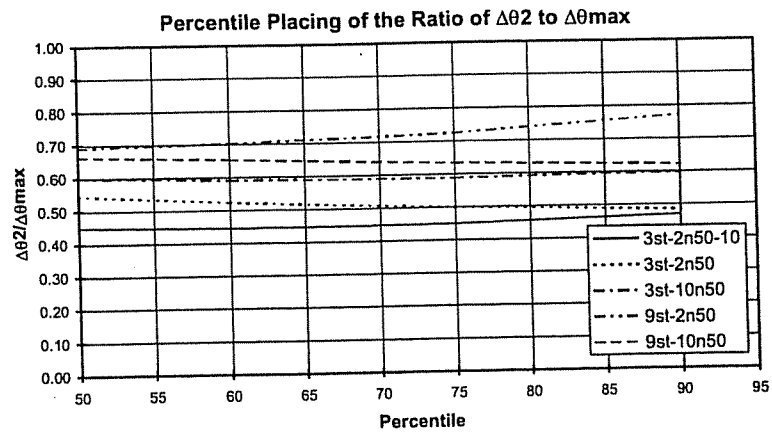


Fig. C.4(b) Statistical Values of Response Parameters

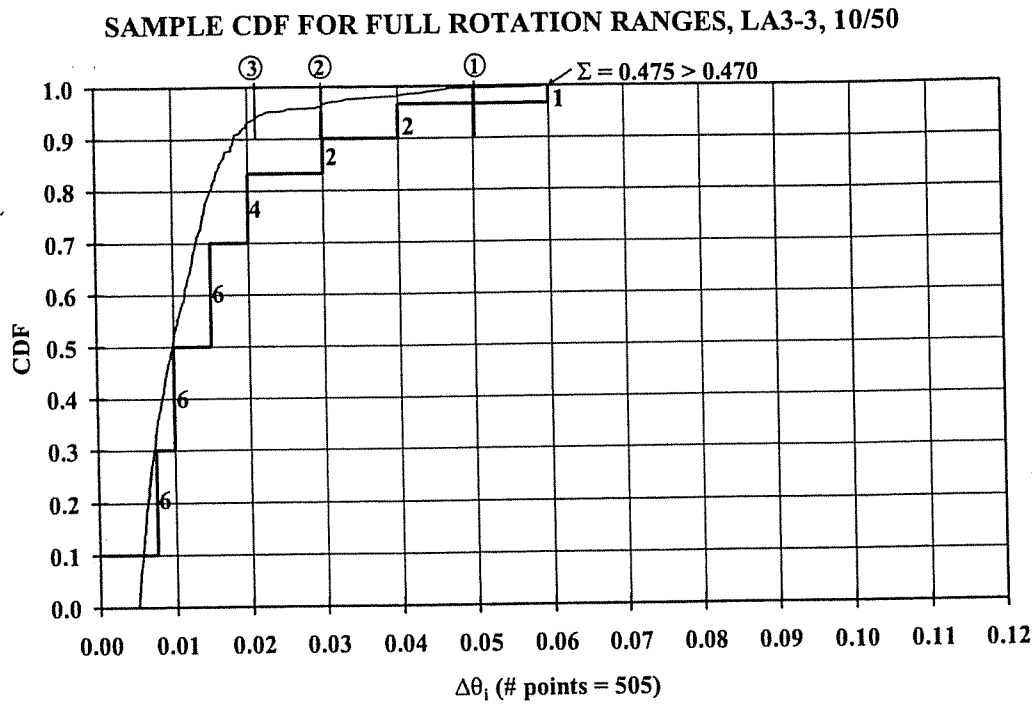


Fig. C.5(a) Matching of Loading History with CDF of 3rd Story of LA3, 10/50 Record Set

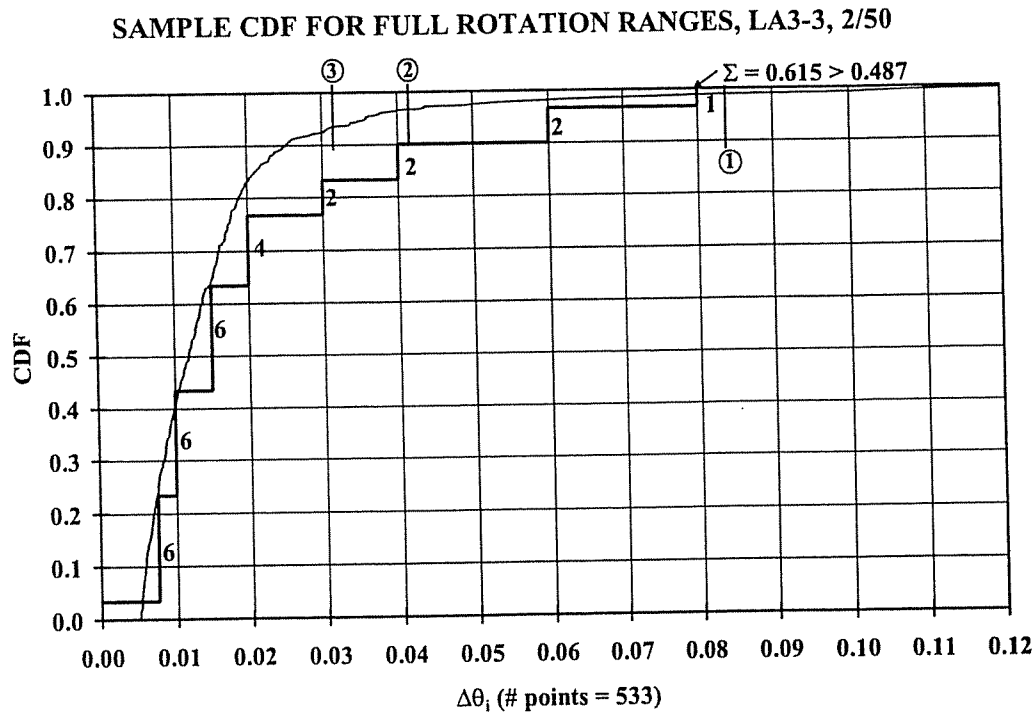


Fig. C.5(b) Matching of Loading History with CDF of 3rd Story of LA3, 2/50-20 Record Set

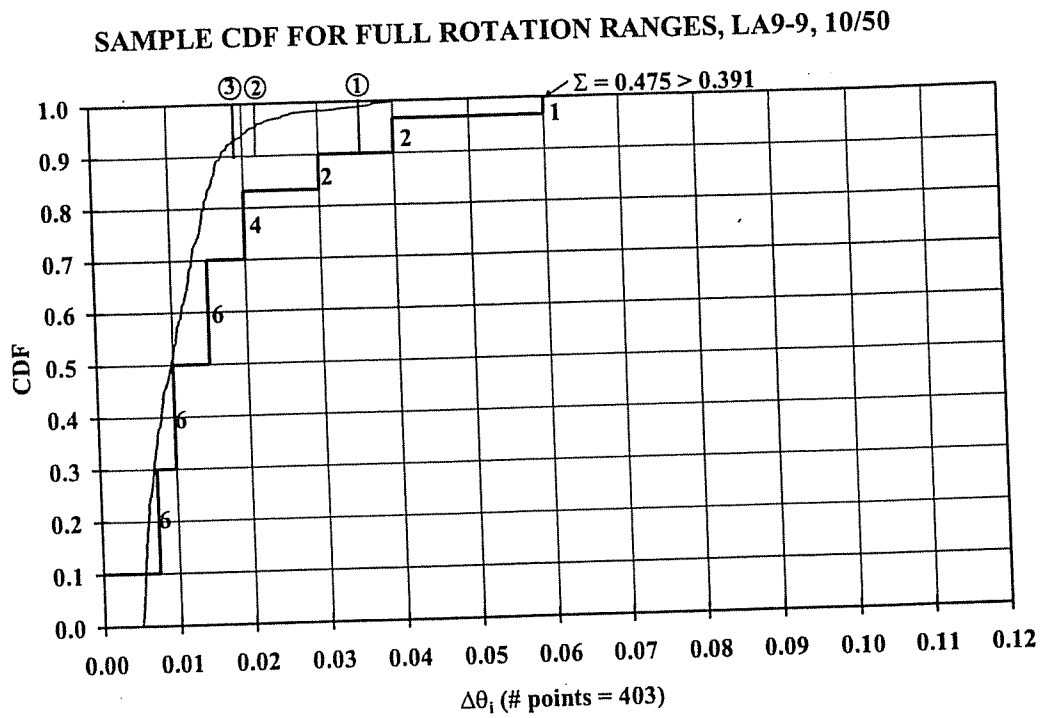


Fig. C.5(c) Matching of Loading History with CDF of 9th Story of LA9, 10/50 Record Set

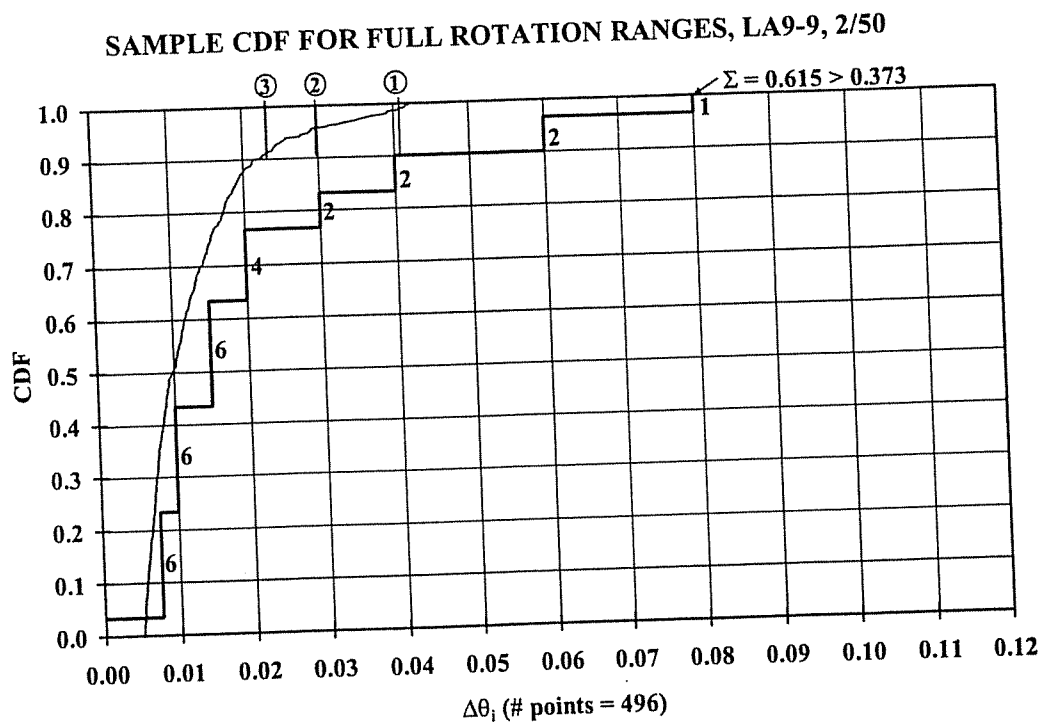


Fig. C.5(d) Matching of Loading History with CDF of 9th Story of LA9, 2/50-20 Record Set

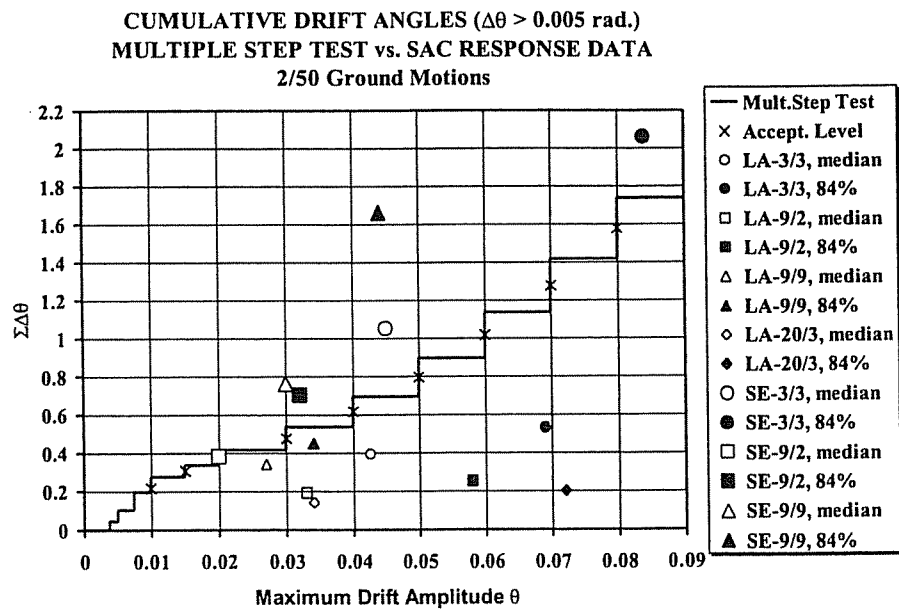


Fig. C6(a) Cumulative Deformations Provided by Loading History Compared to Computed Demands for 2/50 Records

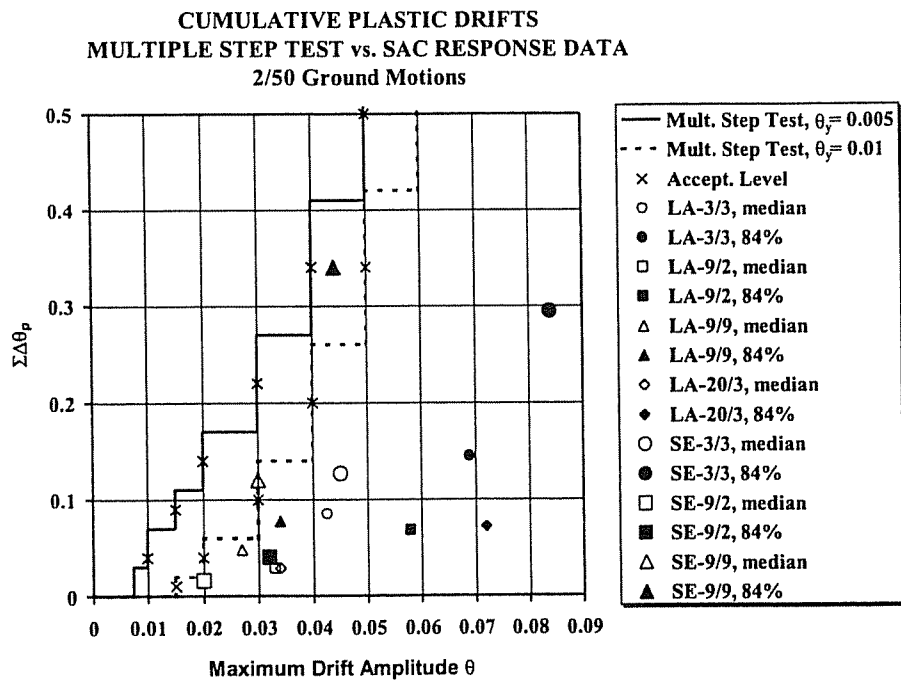


Fig. C6(b) Cumulative Plastic Deformations Provided by Loading History Compared to Computed Demands for 2/50 Records

CUMULATIVE DRIFT ANGLES ($\Delta\theta > 0.005$ rad.)
 MULTIPLE STEP TEST vs. SAC RESPONSE DATA
 10/50 Ground Motions

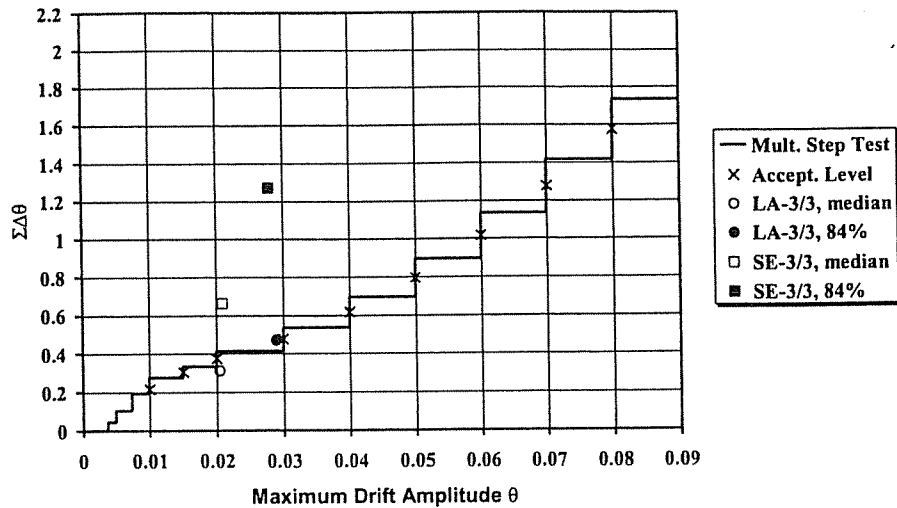


Fig. C6(c) Cumulative Deformations Provided by Loading History Compared to Computed Demands for 10/50 Records

CUMULATIVE PLASTIC DRIFTS
 MULTIPLE STEP TEST vs. SAC RESPONSE DATA
 10/50 Ground Motions

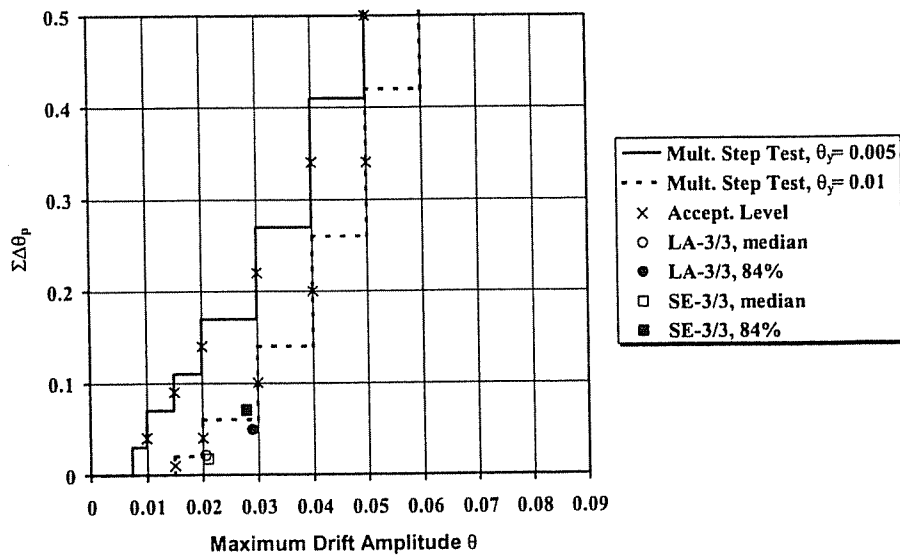


Fig. C6(d) Cumulative Plastic Deformations Provided by Loading History Compared to Computed Demands for 10/50 Records

Cumulative Deformations Phase 1 vs Phase 2 Loading Histories

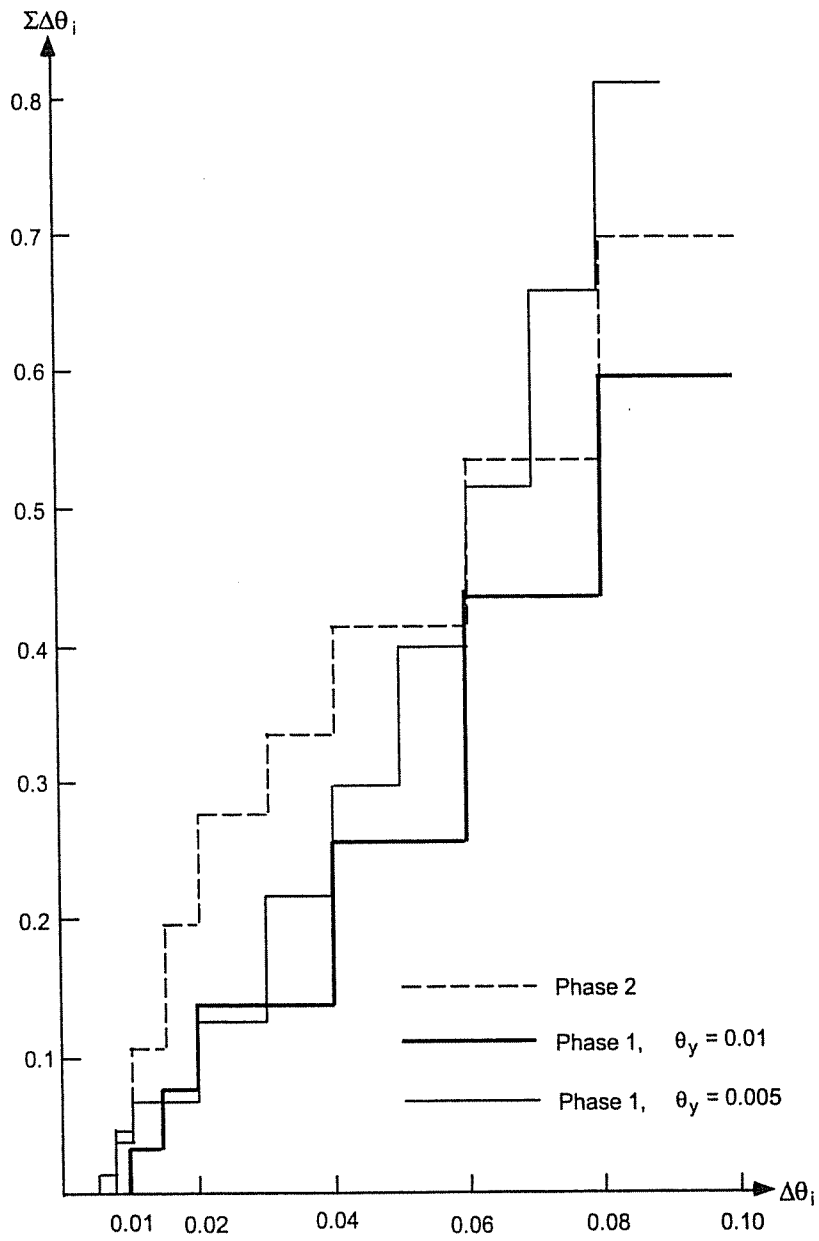


Fig. C.7 Comparison of Cumulative Deformations Between SAC Phase 1 and Phase 2

Loading Histories

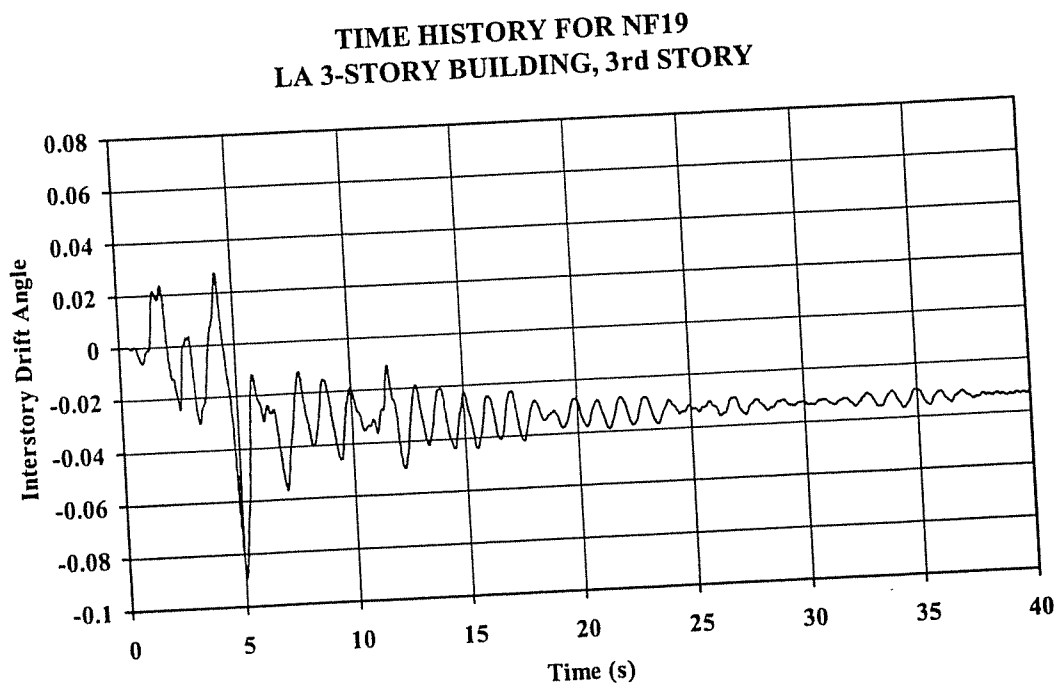
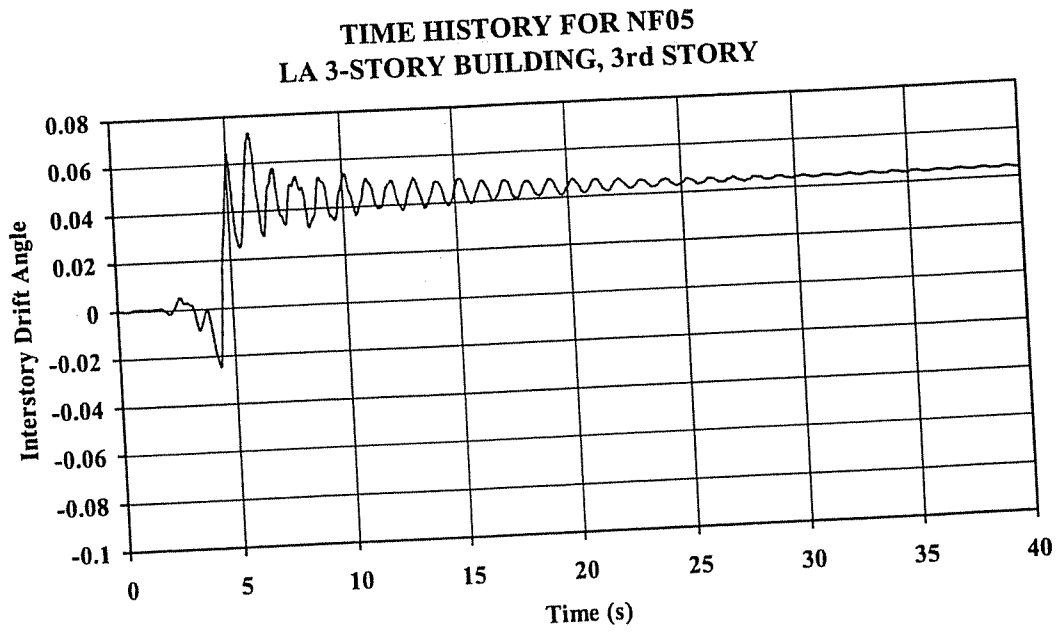


Fig. C.8 Typical Near-Fault Drift Response Histories (LA 3-Story Building, 3rd Story)

LA 3-Story, Model M2

3rd Story

Rainflow Excursions - Ordered from largest to smallest

	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
$\Delta\theta_{max}$	0.0834	0.0973	0.0557	0.0518	0.0793	0.0518	0.0906	0.1169
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):							Loading	
median:			0.0752		stdev of ln:		0.3108	
84th %:			0.1025				History	
							0.08	
	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
$\Delta\theta_2$	0.0683	0.0432	0.0411	0.0207	0.0409	0.0271	0.0810	0.0782
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):							Loading	
median:			0.0453		stdev of ln:		0.4927	
84th %:			0.0741				History	
							0.05	
	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
$\Delta\theta_3$	0.0683	0.0397	0.0268	0.0191	0.0316	0.0264	0.0688	0.0575
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):							Loading	
median:			0.0387		stdev of ln:		0.4842	
84th %:			0.0627				History	
							0.04	

Sum Of Rainflow Excursions $\Sigma\Delta\theta \geq 0.005$

	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
$\Sigma\Delta\theta$	0.7354	0.6382	0.7140	0.2395	0.6513	0.5930	1.2173	1.0910
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
median:		0.6711		stdev of ln:		0.4904		
84th %:		1.0959						

Number of Ranges $N \geq 0.005$

	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
N	48	37	59	16	30	43	62	50
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median:		40.1140		stdev of ln:		0.4407	
	84th %:		62.3313					

Maximum Amplitudes for each Record θ_{max}

Attenuation Coefficients								
	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
θ_{max}	0.0621	0.0726	0.0513	0.0396	0.0539	0.0366	0.0562	0.0892
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
median:			0.0556		stdev of ln:		0.2936	
84th %:			0.0745					
							Loading History 0.06	

Ratio of Max. Rainflow Excursion to Max. Amplitude $\Delta\theta_{max}/\theta_{max}$

Ratio of Peak Maximum		Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):						
	NF03	NF05	NF07	NF09	NF13	NF15	NF17	NF19
$\Delta\theta_{\text{max}}/\theta_{\text{max}}$	1.3433	1.3394	1.0865	1.3078	1.4711	1.4133	1.6115	1.3097
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
		median:		1.3528		stdev of ln:		0.1133
		84th %:		1.5152				
							Loading History 1.33	

Fig. C.9(a) Drift Response Statistics (LA 3-Story Building, 3rd Story)

LA 9-Story, Model M2

4th Story

Maximum Rainflow Excursions, $\Delta\theta_{max}$

	NF3	NF5	NF7	NF9	NF13	NF15	NF17	NF19
$\Delta\theta_{max}$	0.0958	0.0561	0.0483	0.0457	0.0362	0.0526	0.0450	0.0889
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median: 0.0555			stdev of ln: 0.3393				
	84th %: 0.0780							

Maximum Amplitudes for each Record, θ_{max}

	NF3	NF5	NF7	NF9	NF13	NF15	NF17	NF19
θ_{max}	0.0855	0.0486	0.0417	0.0249	0.0286	0.0334	0.0351	0.0494
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median: 0.0404			stdev of ln: 0.3864				
	84th %: 0.0595							

Residual, R

	NF3	NF5	NF7	NF9	NF13	NF15	NF17	NF19
R	0.0695	0.0280	0.0205	0.0030	0.0070	0.0120	0.0100	0.0200
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median: 0.0146			stdev of ln: 0.9487				
	84th %: 0.0376							

8th Story

Maximum Rainflow Excursions, $\Delta\theta_{max}$

	NF3	NF5	NF7	NF9	NF13	NF15	NF17	NF19
$\Delta\theta_{max}$	0.0579	0.0533	0.0362	0.0400	0.0477	0.0393	0.0724	0.0679
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median: 0.0503			stdev of ln: 0.2592				
	84th %: 0.0652							

Maximum Amplitudes for each Record, θ_{max}

	NF3	NF5	NF7	NF9	NF13	NF15	NF17	NF19
θ_{max}	0.0323	0.0303	0.0302	0.0283	0.0350	0.0249	0.0532	0.0464
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median: 0.0340			stdev of ln: 0.2563				
	84th %: 0.0440							

Residual, R

	NF3	NF5	NF7	NF9	NF13	NF15	NF17	NF19
R	0.0082	0.0080	0.0085	0.0090	0.0098	0.0075	0.0240	0.0290
Pulse Type (3, 5, 7, 9, 13, 15, 17, 19):								
	median: 0.0112			stdev of ln: 0.5345				
	84th %: 0.0192							

Fig. C.9(b) Drift Response Statistics (LA 9-Story Building)

LA 3-Story, Model M2

3rd Story

Rainflow Excursions in large pulse direction, $\Delta\theta$ - Ordered from largest to sm

$\Delta\theta_{max}$	NF3 0.0834	NF5 0.0973	NF7 0.0557	NF9 0.0518	NF13 0.0793	NF15 0.0518	NF17 0.0906	NF19 0.1169
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0752 stdev of ln: 0.3108 84th %: 0.1025						Loading History 0.08	
$\Delta\theta_2$	NF3 0.0683	NF5 0.0397	NF7 0.0268	NF9 0.0191	NF13 0.0316	NF15 0.0264	NF17 0.0660	NF19 0.0535
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0377 stdev of ln: 0.4675 84th %: 0.0602						Loading History 0.04	
$\Delta\theta_3$	NF3 0.0316	NF5 0.0279	NF7 0.0215	NF9 0.0155	NF13 0.0313	NF15 0.0193	NF17 0.0477	NF19 0.0445
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0280 stdev of ln: 0.3934 84th %: 0.0415						Loading History 0.03	
$\Delta\theta_4$	NF3 0.0220	NF5 0.0215	NF7 0.0200	NF9 0.0152	NF13 0.0256	NF15 0.0190	NF17 0.0372	NF19 0.0392
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0238 stdev of ln: 0.3279 84th %: 0.0330						Loading History 0.03	
$\Delta\theta_5$	NF3 0.0184	NF5 0.0201	NF7 0.0177	NF9 0.0101	NF13 0.0215	NF15 0.0187	NF17 0.0276	NF19 0.0341
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0200 stdev of ln: 0.3552 84th %: 0.0285						Loading History 0.02	
$\Delta\theta_6$	NF3 0.0164	NF5 0.0180	NF7 0.0167	NF9 0.0073	NF13 0.0203	NF15 0.0174	NF17 0.0251	NF19 0.0280
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0175 stdev of ln: 0.4058 84th %: 0.0263						Loading History 0.02	
$\Delta\theta_7$	NF3 0.0132	NF5 0.0149	NF7 0.0141	NF9 0.0065	NF13 0.0197	NF15 0.0141	NF17 0.0249	NF19 0.0255
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0154 stdev of ln: 0.4351 84th %: 0.0238						Loading History 0.02	
$\Delta\theta_8$	NF3 0.0122	NF5 0.0130	NF7 0.0136	NF9 0.0058	NF13 0.0177	NF15 0.0139	NF17 0.0219	NF19 0.0252
	Pulse Type (3, 5, 7, 9, 13, 15, 17, 19): median: 0.0142 stdev of ln: 0.4495 84th %: 0.0223						Loading History 0.02	

Fig. C.10 Response Statistics for Drift Ranges in Pulse Direction (LA 3-Story Bldg, 3rd Story)

Appendix A

A1 Rationalizing the Use of Interstory Drift Angle as Basic Demand Parameter

Global deformation demand parameter for frame structure:

Roof (top) displacement of frame, δ_t

Best choice for basic demand parameter for frame structure:

Interstory drift, δ_s

Definition: Interstory drift angle $\theta = \delta_s/h$

This is a definition; it is used for convenience only! The joint rotation angle is different from the interstory drift angle because the joint panel zone shear distortion does not cause a joint rotation but contributes to the interstory drift angle.

The interstory drift angle θ is the most stable "local" demand parameter. Anything more local (beam or column plastic hinge rotation, or joint shear distortion) is unstable since the element demands depend on relative element strength and stiffness. In the extreme:

$$\theta_c = \theta, \theta_b = 0, \gamma_p = 0$$

$$\theta_c = 0, \theta_b = \theta, \gamma_p = 0$$

$$\theta_c = 0, \theta_b = 0, \gamma_p = \theta$$

The interstory drift angle θ includes elastic as well as plastic components. The relative values of elastic and plastic components depend on strength and stiffness properties and configuration (bay width versus story height, boundary conditions, etc.).

An evaluation of deformation demands can be accomplished through the following steps:

1. Estimate roof displacement. This can be achieved with reasonable accuracy from SDOF inelastic displacement demand spectra and MDOF modifications. For regular frames much information exists already (Gupta and Krawinkler, 1999).
2. Estimate maximum interstory drift from roof displacement. The relationship between max. δ_s and roof displacement depends on MDOF effects and relative story strength and stiffness. Some statistical information exists (Gupta and Krawinkler, 1999).
3. As defined above: $\theta = \delta_s/h$.
4. The decomposition of θ into element deformation demands needs to be done on a case by case basis. General guidelines for this process are provided in Gupta and Krawinkler, 1999.

Definition of Interstory Drift Angle in Test Specimens:

If column is pinned and beam deflects vertically by δ_b : $\theta = \delta_b/L$

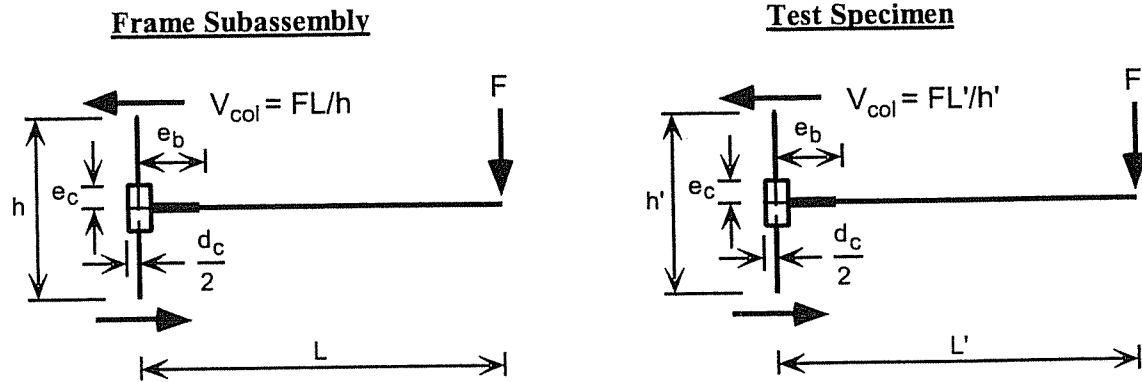
If beam is pinned and column deflects horizontally by δ_c : $\theta = \delta_c/h$

In a beam-column assembly test specimen, the elastic component of θ is rather sensitive to the specimen geometry (primarily beam span). Thus, the beam length should be equal to half the bay

width. The plastic component of θ is less sensitive to geometry, but its definition may be ambiguous because of the need to define yield rotation. Thus, it appears to be the best choice to use the total (elastic plus plastic) interstory drift angle as basic demand parameter. The components of the interstory drift angle for a frame substructure and the corresponding single beam test specimen are identified on the following pages. It is assumed that the substructure consists of a column, a beam portion of uniform cross-section, possibly a beam portion with increased stiffness (e.g., a haunch) [in the following equations it is assumed that this portion is very stiff, i.e., rigid], and a joint panel zone.

A2 Deformation Control for Experimental Studies

Geometry and Loading Parameters



Assumptions:

- One column - one beam (or two beams) test specimen.
- Column hinged at top and bottom, representing points of inflection at midheight of adjacent stories in frame, i.e., column height h' is equal to frame story height h .
- Beam length L' should represent distance from column centerline to point of inflection in frame beam (i.e., half of bay width, L).
- Point of inflection in frame beam does not deflect vertically.
- Important deformation components (elastic and plastic) are
 - flexural deformations in column outside joint panel zone
 - flex. deformations in beams outside panel zone and outside "rigid" beam portion
 - shear deformations within joint panel zone
- Plastic deformations may occur at the following plastic hinge locations:
 - In column just outside joint panel zone (distance e_c from beam centerline)
 - In beam at a distance e_b from column centerline (at column face or a distance away from column face)
 - In joint panel zone

Force and Deformation Control Parameters:

- Applied force at beam end, F
- Deflection at beam end, δ'

Force and Deformation Evaluation Parameters:

- Column shear force, V_{col}
- Joint rotation, $\theta' = \delta'/L'$
 $[\theta' = \theta, \text{ if } L' = L \text{ and } h' = h \text{ (see following pages)}]$.

Observations:

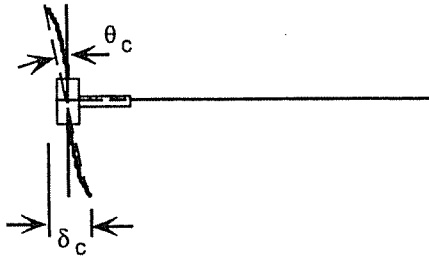
- The relationships between F and V_{col} and between θ' and δ' will not accurately simulate frame conditions unless the column heights in test specimen and frame are equal and the specimen beam length L' is equal to the distance from the column centerline to the frame beam point of inflection, L (see equations on following pages).
- The effect of gravity loading, which will lead to a variation of the beam M/V ratio in the frame beam (i.e., a variation of the moment gradient), will not be simulated in the experiment unless a second load is applied in the beam span.
- The deflection components and corresponding story drifts for the frame substructure and the test specimen are as shown on the following pages.

A3 Deflection (Drift) Components

(a) Due to Flexural Deformations in Column

Elastic Behavior:

Frame Subassembly



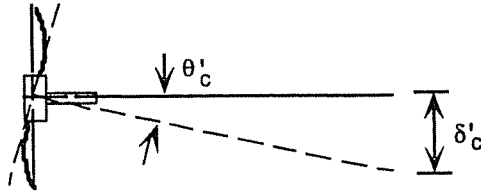
$$\delta_c = \frac{(h - 2e_c)^3}{12EI_c} V_{col}$$

$$\theta_c = \frac{(h - 2e_c)^3}{12EI_c} \frac{V_{col}}{h}$$

$$\theta_c = \frac{(h - 2e_c)^3}{12EI_c} \frac{FL}{h^2}$$

Thus, $\theta'_c = \theta_c$ if $L' = L$ and $h' = h$.

Test Specimen



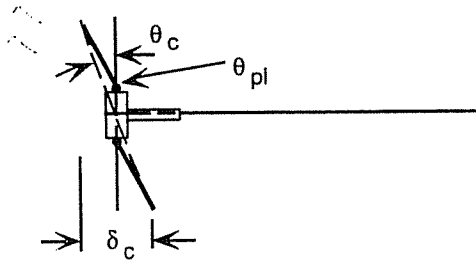
$$\delta'_c = \frac{(h' - 2e_c)^3}{12EI_c} \frac{L'}{h'} V_{col}$$

$$\theta'_c = \frac{(h' - 2e_c)^3}{12EI_c} \frac{V_{col}}{h'}$$

$$\theta'_c = \frac{(h' - 2e_c)^3}{12EI_c} \frac{FL'}{h'^2}$$

Inelastic Behavior:

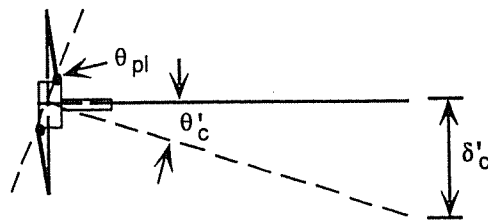
Frame Subassembly



$$\delta_c = (h - 2e_c) \theta_{pl}$$

$$\theta_c = \frac{(h - 2e_c)}{h} \theta_{pl}$$

Test Specimen



$$\delta'_c = \frac{L'(h' - 2e_c)}{h'} \theta_{pl}$$

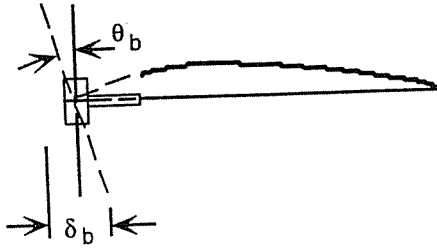
$$\theta'_c = \frac{(h' - 2e_c)}{h'} \theta_{pl}$$

(b) Due to Flexural Deformations in Beam

Example: Flexural deformations within distance e_b are neglected.

Elastic Behavior:

Frame Subassembly

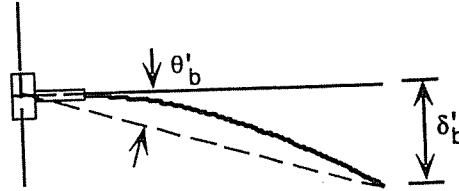


$$\delta_b = \frac{(L - e_b)^3}{3EI_b} \frac{h}{L} F$$

$$\theta_b = \frac{(L - e_b)^3}{3EI_b} \frac{F}{L}$$

$$\theta_b = \frac{(L - e_b)^3}{3EI_b} \frac{h}{L^2} V_{col}$$

Test Specimen



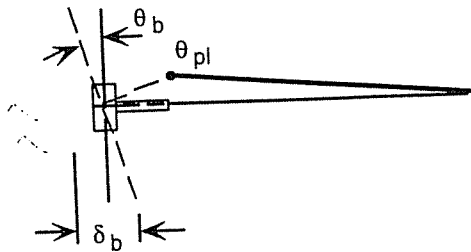
$$\delta'_b = \frac{(L' - e_b)^3}{3EI_b} F$$

$$\theta'_b = \frac{(L' - e_b)^3}{3EI_b} \frac{F}{L'}$$

$$\theta'_b = \frac{(L' - e_b)^3}{3EI_b} \frac{h}{L'^2} V_{col}$$

Inelastic Behavior:

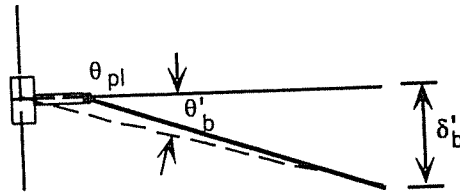
Frame Subassembly



$$\delta_b = \frac{h(L - e_b)}{L} \theta_{pl}$$

$$\theta_b = \frac{L - e_b}{L} \theta_{pl}$$

Test Specimen



$$\delta'_b = (L' - e_b) \theta_{pl}$$

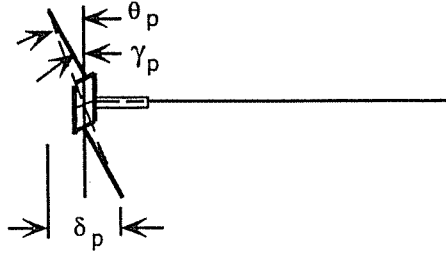
$$\theta'_b = \frac{L' - e_b}{L'} \theta_{pl}$$

Note that θ_{pl} is significantly larger than θ_b if e_b is large (improved connection).

(c) Due to Shear Deformations in Joint Panel Zone

Elastic Behavior:

Frame Subassembly



$$\delta_p = (h - 2e_c) \gamma_p$$

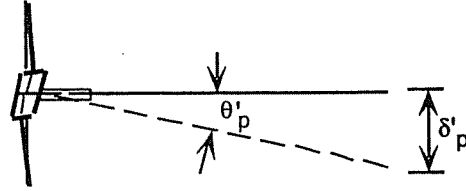
$$\theta_p = \frac{(h - 2e_c)}{h} \gamma_p$$

$$\gamma_p = \frac{V_p}{A_s G}$$

$$V_p \equiv \frac{M}{2e_c} - V_{col} \equiv \frac{FL}{2e_c} - \frac{FL}{h} = FL \left(\frac{1}{2e_c} - \frac{1}{h} \right)$$

$$\theta_p \equiv \frac{(h - 2e_c)}{h} \frac{FL}{A_s G} \left(\frac{1}{2e_c} - \frac{1}{h} \right)$$

Test Specimen



$$\delta'_p = \frac{L'(h' - 2e_c)}{h'} \gamma_p$$

$$\theta'_p = \frac{(h' - 2e_c)}{h'} \gamma_p$$

Inelastic Behavior:

The general equations for δ_p and θ_p (first two lines) hold true.

From the test results the individual plastic components can be evaluated (requires proper instrumentation), and the total plastic interstory drift angle can be deduced directly from the F - δ' diagram.

Appendix B – The Need for Cycle Counting

Cycle counting is needed in order to convert the response time history of the interstory drift angle into a series of cycles from which a loading history can be rationalized. A cycle is defined as a closed unit with equal deformation ranges in the two directions. A cycle may have a mean deformation, i.e., it need not be symmetric w.r.t. the origin. It is assumed that damage is proportional to the deformation range of the cycle, but it is also understood that large mean deformations may have a significant effect on damage accumulation. In the basic loading history, mean effects are not considered.

A beam to column connection may deteriorate in positive or negative bending (tension in bottom or top flange of beam). A single excursion will cause damage primarily in one direction (tension in case of weld fracture mode, or compression in flange buckling mode). If only one test is to be performed, there is no strong argument to favor one direction of loading over the other since all deterioration modes need to be evaluated. Thus, loading should be done with equal emphasis in both directions, i.e., with symmetric cycles. Even if a pulse type response is observed (large excursion in one direction), this criterion applies in order to test behavior in positive as well as negative bending.

Basic considerations for cycle counting:

1. EQ time history response does not contain cycles; it contains response excursions that follow a pattern controlled by the characteristics of the ground motion and of the structure.
2. In a damage context, it is hypothesized that damage accumulation follows the following pattern:
Damage is proportional to deformation range (better: plastic deformation range) raised to a power. In the simplest case the power is 1.0, in which case the cumulative damage is proportional to the sum of ranges. More realistically, the power is larger than 1.0 (1.5 to 2.0 for steel and plastic behavior). Thus, a larger range counts much more than a smaller range.
3. As a consequence, it is critical to arrange the response time history in a manner that creates the largest possible ranges. This implies that smaller excursions need to be considered as interruptions of larger ones.
4. The rainflow cycle counting method achieves the objective outlined in 3. There is a basic rainflow cycle counting method and a simplified one.
5. In the simplified rainflow cycle counting method the time history response is re-arranged in a manner such that the history always starts with the maximum amplitude point (i.e., the portion of the history following the maximum amplitude point is moved to the front of the history). The end of the moved-up portion is connected artificially to the beginning of the time history. If there is little or no residual deformation, this amounts to a short artificial connection; however, if there is a large residual deformation, the artificial connection amounts to adding an excursion that may cause significant damage. As a consequence, the damage due to the

largest excursion will be represented realistically in one direction but will be overestimated in the other direction after rainflow cycle counting, compared to the original history. This is a necessary compromise since the direction in which the largest excursion will occur is not known.

6. If the simplified rainflow cycle counting method is used, the resulting history will only have full cycles. This is the case in the approach used here for the basic loading history. Therefore, all the results presented on this loading history are in terms of cycles and not excursions.

Appendix C – Summary of Statistical Information

Results are presented from dynamic time history results of LA 3-story and 9-story buildings, using the fault normal and fault parallel components of the LA 2/50 and 10/50 record sets. Analysis was performed with a 2-D centerline model of the NS perimeter frames, with plastic hinging permitted only at the ends of beams and columns. Joint panel zone strength and stiffness are not represented in this model.

The parameter represented is the interstory drift angle θ .

	Nt	θ_{max}	$\Delta\theta_{max}$	$\Delta\theta_2$	$\Delta\theta_3$	$\Sigma\Delta\theta_i$	Np	$\Sigma\Delta\theta_{pi}$
SAMPLE								
Median:	23.5	0.0230	0.0356	0.0206	0.0182	0.279	6	0.0404
Mean:	25.3	0.0234	0.0363	0.0223	0.0181	0.293	7.0	0.0426
COV:	0.409	0.317	0.277	0.237	0.110	0.366	0.400	0.449
LOGNORMAL DISTRIBUTION								
Slope:	1.931					3.9443	8.0179	2.3845
Intercept:	-6.0617					15.096	32.228	2.3405
Median:	23.1					0.0218	0.0180	0.275
75th Percentile:	32.7					0.0258	0.0195	0.365
90th Percentile:	44.8					0.0301	0.0211	0.470
WEIBULL DISTRIBUTION								
Slope:		3.4521	3.6974					
Intercept:		12.594	11.884					
Median:		0.0234	0.0364					
75th Percentile:		0.0286	0.0439					
90th Percentile:		0.0332	0.0504					

Note: "Slope" and "Intercept" are taken from the best fit line in the plot of the inverse cumulative distribution (for specified distribution) versus the given response parameter.

Fig. A.3.1 Statistics for 3rd Story of 3-Story LA Building, 10/50 Record Set

	Nt	θ_{max}	$\Delta\theta_{max}$	$\Delta\theta_2$	$\Delta\theta_3$	$\Sigma\Delta\theta_i$	Np	$\Sigma\Delta\theta_{pi}$
SAMPLE								
Median:	24.5	0.0318	0.0503	0.0314	0.0253	0.394	10	0.102
Mean:	26.7	0.0395	0.0629	0.0347	0.0272	0.402	10.8	0.116
COV:	0.329	0.517	0.494	0.333	0.269	0.303	0.362	0.469
LOGNORMAL DISTRIBUTION								
Slope:	2.6799					2.959	3.5938	2.8648
Intercept:	-8.665					10.072	13.054	2.7372
Median:	25.4					0.0332	0.0265	0.385
75th Percentile:	32.6					0.0418	0.0319	0.487
90th Percentile:	40.9					0.0513	0.0378	0.602
WEIBULL DISTRIBUTION								
Slope:		2.1788	2.1611					
Intercept:		6.7591	5.6821					
Median:		0.0380	0.0609					
75th Percentile:		0.0522	0.0839					
90th Percentile:		0.0659	0.1061					

Note: "Slope" and "Intercept" are taken from the best fit line in the plot of the inverse cumulative distribution (for specified distribution) versus the given response parameter.

Fig. A.3.2 Statistics for 3rd Story of 3-Story LA Building, 2/50-20 Record Set

SAMPLE

LOGNORMAL DISTRIBUTION

WEIBULL DISTRIBUTION

Fig. A.3.3 Statistics for 9th Story of 9-Story LA Building, 10/50 Record Set

SAMPLE

LOGNORMAL DISTRIBUTION

WEIBULL DISTRIBUTION

Fig. A.3.4 Statistics for 9th Story of 9-Story LA Building, 2/50-20 Record Set

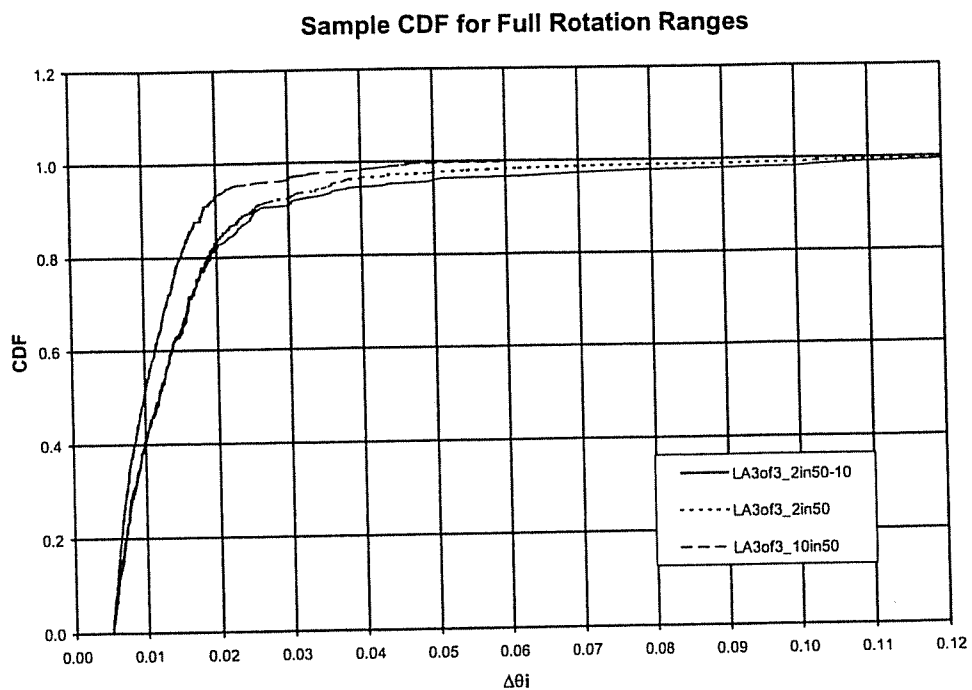


Fig. A.3.5 Sample CDFs of all $\Delta\theta_i$ Values, 3rd Story of 3 Story LA Building, 10/50 Records, 2/50 Records (all 20 records), and 2/50 Records (10 fault normal records)

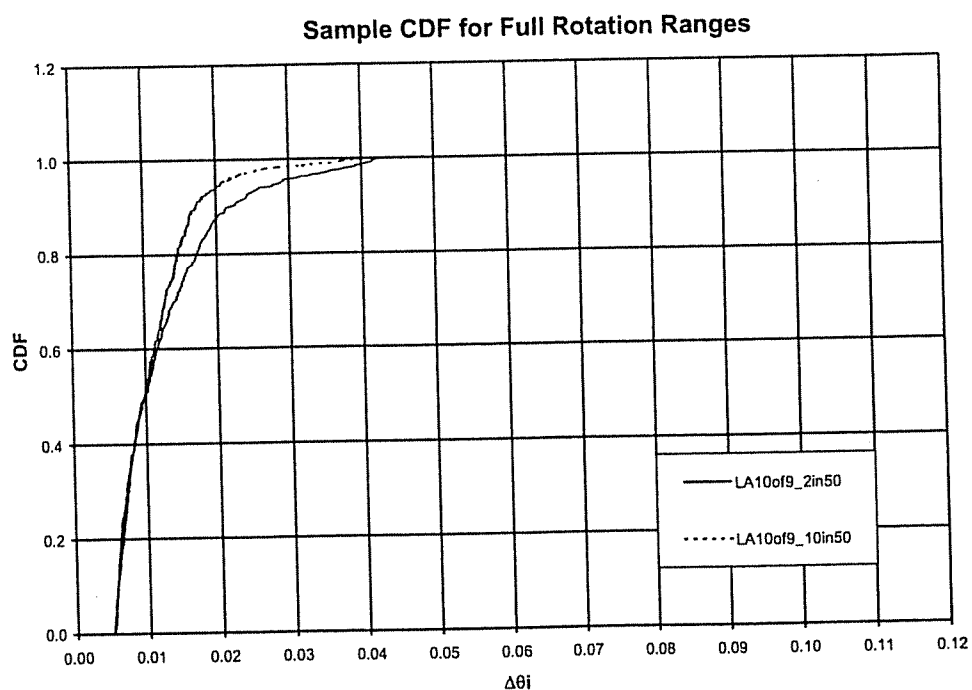


Fig. A.3.5 Sample CDFs of all $\Delta\theta_i$ Values, 9th Story of 9 Story LA Building, 10/50 Records, and 2/50 Records (all 20 records)