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Performance Assessment of Steel Components

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This paper presents a methodology for the assessment of cumulative damage in structural steel components subjected to cyclic inelastic loading histories of the type experienced in earthquakes. The methodology is based on low-cycle fatigue concepts and the hypothesis of linear damage accumulation. It will be shown that seismic performance of a component depends on two structural performance parameters and on the number and amplitudes of all inelastic excursions and not only on the maximum excursion. Experimental and analytical procedures for obtaining the parameters needed for a performance assessment are suggested in the paper.

INTRODUCTION

Routine seismic design is based on a simplified approach in which "brittle" failure modes are, at least to some degree, prevented through proper strength design of individual elements and connections. For instance, buckling of vertical load carrying columns can be prevented by designing columns for the maximum axial forces that can be transferred to them by the other elements of the lateral load resisting system. Also, brittle connection failure can be prevented by designing connections to be stronger than the connected elements. Thus, "brittle" failure modes, which are characterized by a rapid loss in resistance, can be avoided through proper strength design.

There are, however, a number of other "failure" modes the structure may experience in the process of dissipating energy through inelastic deformation. In the design process these modes, which could be referred to as deterioration modes, are considered implicitely through proper detailing. Typical examples of deterioration modes in steel structures are local and lateral torsional buckling, crack propagation at weldments and at reduced cross sections (e.g., net sections at bolt holes), shear buckling in joint panel zones and link beams, and buckling of bracing members in braced frames. Much research has been performed on these deterioration modes in order to develop detailing criteria (e.g., b/t limitations for flange buckling, lateral bracing requirements, etc.) whose purpose it should be to limit deterioration in a severe earthquake to an acceptable value.

The issue that has not received much attention is that deterioration is a function of loading history. In essence, every cycle in a loading history causes damage and, even though this damage may not cause noticeable deterioration, it will affect the onset and rate of deterioration at a later time. In elements that undergo severe inelastic deformations, the amount of damage caused by elastic cycles is usually negligible and only inelastic cycles need

This paper, with minor modifications, is a reprint of a paper published in the Proceedings of the Pacific Structural Steel Conference, Auckland, New Zealand, August 4-8, 1986.

to be considered. Thus, for a realistic assessment of deterioration, all inelastic cycles and their cumulative effect on damage should be accounted for.

In the past almost all experimentation on deterioration behavior has been limited to single specimen testing. That is, only one specimen of a kind is tested, using a preselected and subjective loading history. There is no procedure known today by which the results from such a single test can be generalized to other loading histories without prior knowledge of deterioration characteristics. Thus, from single specimen tests a reliable performance assessment cannot be achieved. In some cases, loading histories are prescribed by standard documents (e.g., Standards Association of New Zealand, 1976), which permits a relative but not absolute performance assessment.

This paper discusses the issue of performance assessment of deteriorating structural components from several viewpoints. The following questions are addressed. Is it necessary to consider cumulative damage in performance assessment? Which simple models can be employed to capture the important aspects of deterioration under arbitrary loading histories? What information is needed to evaluate seismic performance of deteriorating structural components? How can this information be obtained, i.e., what experiments are needed to evaluate cyclic deformation capacity in the presence of deterioration and what analytical approaches are needed to determine the cyclic deformation demand in severe earthquakes? The convolution of capacity and demand will be a measure of seismic performance.

CUMULATIVE DAMAGE MODEL

The task at hand is to find a damage function that accounts for the cumulative effects of cycles of different magnitudes on the present state of resistance as well as the closeness to failure of a structural component. Failure in this context is defined as an unacceptable limit state which may be associated with onset of deterioration, an unacceptable decrease in resistance, or complete loss of resistance.

The simplest model for this purpose is the classical low-cycle fatigue model for metallic materials. In this model it is postulated that under constant amplitude cycling the number of cycles to failure, N_f, can be related to the plastic deformation range of the cycle, $\Delta\delta_p$, by an equation of the type

$$N_f = C^{-1}(\Delta\delta_p)^{-c} \qquad (1)$$

where C and c are structural performance parameters. The plastic deformation range $\Delta\delta_p$ is defined in Fig. 1. The deformation quantity to be used in this equation could be a strain, angle of distortion, rotation, deflection, or any other deformation quantity, depending on the failure mode under study.

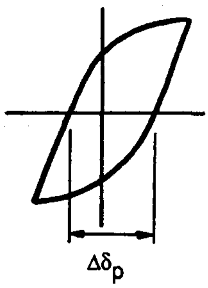


Figure 1 - Plastic deformation range

Using the hypothesis of linear damage accumulation (Miner's rule), the damage per cycle of magnitude $\Delta\delta_p$ is $1/N_p$ and the accumulated damage after N cycles of different magnitudes $\Delta\delta_{pi}$ is then given as

$$D = \sum_{i=1}^{N} \frac{1}{N_{fi}} = C \sum_{i=1}^{N} (\Delta\delta_{pi})^{c} \tag{2}$$

This equation is the simplest damage model that can be proposed for structural components. The use of Miner's rule has the shortcomings that mean deformation and sequence effects are not considered. These effects could be included through modifications to the model. This is not done here in order to keep the model simple and because these effects are, according to preliminary test results, in many cases not very important for damage accumulation in highly inelastic cycles.

In order to use this model, individual excursions from a seismic response history must be converted into as many closed cycles as possible since the baseline data for Eq. 1 are obtained from constant amplitude tests and damage is counted per cycle and not per excursion. This can be achieved through cycle counting methods of which the rain-flow method is the most widely used one for low-cycle fatigue problems. The basic idea behind this method is to treat small excursions as interruptions of larger ones and to match the highest peak and deepest valley, then the next highest and deepest, etc., until all peaks and valleys of a cyclic loading history have been paired. Details of this method and of other important aspects of damage modeling are discussed by Krawinkler et al., 1983.

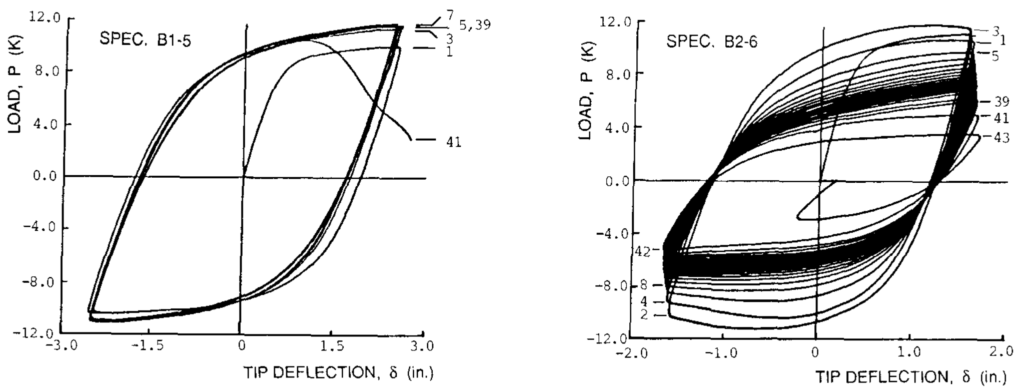
If the cumulative damage model given by Eq. 2 were accurate, a limit value of damage of unity would constitute failure. Because of the shortcomings of Miner's rule and the hypothetical nature of Eq. 1, the limit value of damage associated with failure will be a random variable denoted here by γ. A probabilistic assessment of component failure can then written as

$$P_f = P\,[D > \gamma] = P\,[C \sum_{i=1}^{N} (\Delta\delta_{pi})^{c} > \gamma] \tag{3}$$

No attempts are made here to pursue this probabilistic formulation because of insufficient data on the variables. But both Eqs. 2 and 3 show that performance assessment will necessitate two sets of data. One to determine the structural performance parameters C and c, and the other to determine the individual plastic deformation ranges $(\Delta\delta_{pi})$ and the number of inelastic cycles (N) that the component may experience in an earthquake. The following discussion will illustrate how this information can be obtained and it will be shown that the simple damage model given by Eq. 2 gives reasonable results for specific failure modes in components of steel structures.

The two failure modes being studied are crack propagation at a beam-to-column connection weld and local buckling in beam flanges. The rate of deterioration in these two modes differs considerably as shown in Fig. 2. Figure 2(a) shows the response of a cantilever beam that failed by weld fracture at the connection weld. The hysteresis loops of this test specimen are stable until fracture occurs. Weld fracture is preceded by cyclic crack propagation that causes damage but little or no noticeable deterioration in the element response. In this case the deterioration threshold life is long and deterioration occurs rapidly once a crack has grown to a critical size. Quite differently, if the beam response is governed by local buckling (see Fig. 2(b)), deterioration occurs early in the history and occupies most of the useful life of the specimen. Thus, in the first case the deterioration threshold range needs to be modeled whereas in the second case emphasis is on modeling of the deterioration range.

The deterioration behavior studied in these two failure mode is characteristic for most localized "ductile" failure modes in steel structures. Thus, the models proposed here can in concept be applied also to other failure modes such as lateral torsional buckling and fracture at net sections in bolted connections. They should, however, not be applied to more global failure modes such as member buckling.



(a) Large deterioration threshold (crack prop.) (b) Gradual deterioration (local buckling)

Fig. 2 - Failure modes with different deterioration behavior

DAMAGE MODEL FOR CRACK PROPAGATION AT WELDMENTS

A series of ten identical cantilever specimens of the type shown in Fig. 3 were tested to study the crack propagation and fracture mode of failure. The beam flanges were welded to the flange of a column stub with full penetration butt welds. The cantilever beams were loaded at the tip, with tip deflection being the control parameter for the loading history. Most of the specimens were subjected to constant deflection amplitude cycling in order to assess the validity of Eq. 1 and to determine the structural performance parameters C and c. One test was performed under variable amplitude cycling to examine the accuracy of the cumulative damage model.

In all test specimens, crack propagation at the toe of the beam flange to column flange weld (see Fig. 4) was the cause of deterioration and failure. One or several small cracks near the beam flange centerline were observed very early in the loading history. These cracks joined soon and a single crack propagated through the heat affected zone (HAZ) of the flange with a crack front as shown in Fig. 5. When the crack depth reached about half the flange thickness, fracture occurred through the flange and the through-crack propagated rapidly across the flange width.

The crack growth through the flange thickness did not lead to a noticeable deterioration in the global load-deflection response of the specimens. But rapid deterioration occurred once a through-crack had formed and the crack propagated across the flange width. This rapid deterioration occupies a negligible portion of the life of the specimens and failure can be assumed to coincide with the occurrence of a through-crack in the flange. In this manner only the deterioration threshold range needs to be modeled and the deterioration range is neglected.

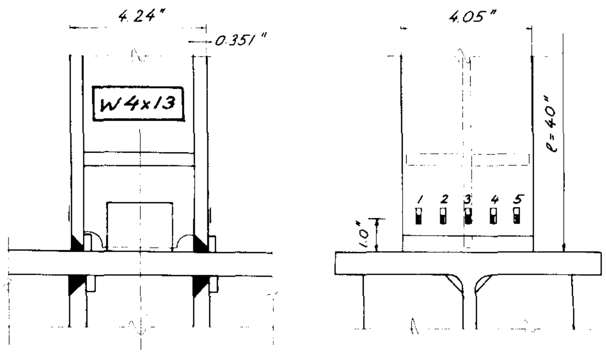


Fig. 3 - Crack propagation specimen

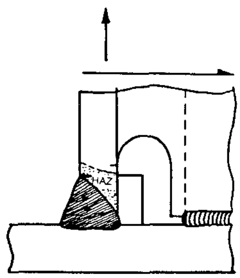


Fig. 4 - Crack at weld toe

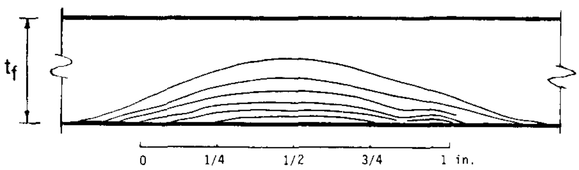


Fig. 5 - Crack shapes at different stages of growth

Baseline data for damage modeling can be obtained from two approaches. One is to use the observed data directly to determine the structural performance parameters C and c in Eq. 1. Taking the measured plastic strain range close to the crack, $\Delta\varepsilon_p$, as the controlling deformation parameter, the constant amplitude tests give the results shown in Fig. 6. With some generosity one can put a straight line to the logarithmic plot of the data points but there is an evident scatter about the regression line which indicates that at least one of the performance parameters must be treated as a random variable.

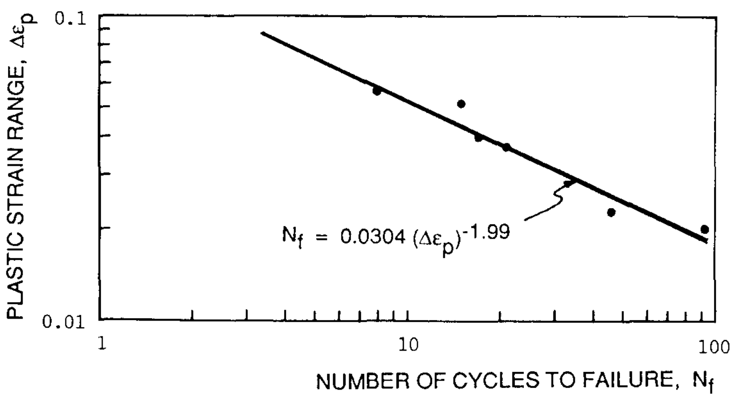


Fig. 6 - Results of constant amplitude tests

An alternative approach is to use fracture mechanics concepts to predict crack growth and to convert a crack growth model into a cumulative damage model. In this approach an initial crack of size a_o is assumed to exist in the virgin specimen because of the presence of imperfections at the weld toe, and crack growth through the flange is predicted from a crack growth rate model. In all constant amplitude tests, crack size measurements taken at advanced stages of cracking (relatively large cracks) have given rather consistent data for a simple crack growth rate model. These measurements, obtained from striations on the fracture surface, have confirmed the following conclusions drawn by others (e.g., Solomon, 1972) for crack propagation in inelastically cycled mild steel. If a specimen is cycled with a constant strain amplitude close to the crack, then the logarithm of the crack depth, a, is related linearly to the number of cycles, N. This is illustrated for one of the constant amplitude tests in Fig. 7. Furthermore, when the slopes of these crack growth vs. N lines are plotted on a log-log paper against the plastic strain ranges of the individual tests, see Fig. 8, it is concluded that for constant amplitude cycling the crack growth rate per cycle, da/dN, and the plastic strain range, $\Delta\varepsilon_p$, can be related by an equation of the form

$$da/dN = \alpha a(\Delta\varepsilon_p)^\beta \tag{4}$$

where a is the crack size and α and β are fracture mechanics parameters that depend on the material properties, the geometry of the specimen, the shape of the crack, and the location at which strains are measured.

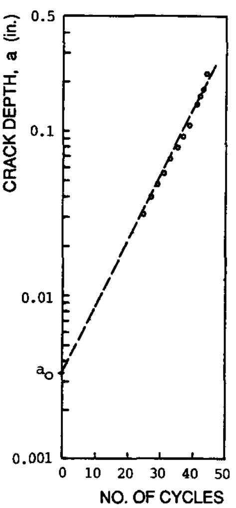


Fig. 7 - Crack growth in a constant amplitude test

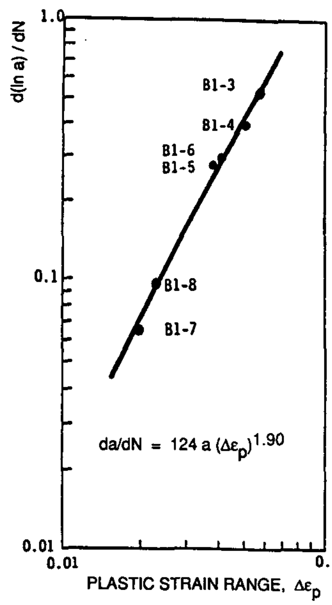


Fig. 8 - Crack growth rate model

Figure 8 shows that the data points deviate relatively little from the regression line, indicating that the scatter is small compared to a later discussed uncertainty, and that it should be acceptable to treat both α and β as deterministic quantities.

Assuming that the initial crack size a_o is known, the number of cycles to failure for constant amplitude cycling can be evaluated by integrating the crack growth rate model from a_o to a critical crack size a_c, resulting in the expression

$$N_f = \alpha^{-1} \ln \frac{a_c}{a_o} (\Delta\varepsilon_p)^{-\beta} \qquad (5)$$

This expression is identical in form to Eq. 1. Thus, the low-cycle fatigue approach and the fracture mechanics approach result in identical formulations for life predictions although the two approaches are based on distinctly different concepts. Moreover, the use of the crack growth rate model given by Eq. 4 for cycles with variable amplitudes is equivalent to the use of Miner's rule in the damage model expressed by Eq. 2. Thus, this damage model can be expressed in terms of fracture mechanics parameters as follows:

$$D = \sum_{i=1}^{N} \frac{1}{N_{fi}} = \alpha \left(\ln \frac{a_c}{a_o}\right)^{-1} \sum_{i=1}^{N} (\Delta\varepsilon_{pi})^{\beta} \qquad (6)$$

This expression is identical in form to Eq. 2 with the following equivalence: $c = \beta$ and $C = \alpha[\ln (a_c/a_o)]^{-1}$. The advantage of the formulation given by this equation is that fracture mechanics parameters are involved in the determination of the structural performance parameters. For one, there is some indication that the exponent β and therefore the exponent c is insensitive to variations in geometry and is usually in the order of 1.9 for mild steel. Perhaps more important, Eq. 6 shows that the performance parameter C depends on the critical and initial crack sizes. For the component tested in this study the critical crack size a_c was approximately equal to half the flange thickness and did not vary much between specimens. Since α is a parameter with little scatter, it is clear that much of the large scatter observed in the tests (see Fig. 6) must be attributed to variations in the initial crack size a_o.

The initial crack size depends strongly on initial imperfections at the weld toe or at any other location at which crack propagation may occur. In life or damage predictions for welded connections this initial crack size will be the most critical quantity. Inaccuracies in the analytical model, such as the neglect of sequence effects, will be small compared to the uncertainties in life predictions caused by differences in initial crack size due to variations in workmanship or other parameters that affect the initial crack size. To complicate the issue, the writer claims that it is most difficult or almost hopeless to measure an initial crack size that can be used reliably in a crack propagation model at a weldment. Initial cracks have ill defined geometry and do not propagate according to fracture mechanics principles until they have formed a well defined crack front.

The inability to measure initial crack size together with the great variations in a_o make the crack propagation and fracture problem a most difficult one and explain the many arguments about the reliability of welded connections. If we could get adequate statistical data on a_o we could settle these arguments through mathematical models of the type given by Eq. 6. One possible way of estimating initial crack size is the procedure used in this study. In this procedure the initial crack size is predicted by using the regressed values of α and β and solving Eq. 5 for a_o for each constant amplitude test. For the specimen used here, the mean value and standard deviation of a_o were found to be 0.00163 in. (0.041 mm) and 0.00123 in. (0.031 mm), respectively, indicating a very large scatter.

Equation 4 was utilized to predict the crack growth in a test with variable amplitude cycling. The history for this test was a realistic seismic response history that was applied repeatedly to the specimen. The results of the experiment and of two analytical predictions, one based on the mean of a_o and the other on mean-σ of a_o are shown in Fig. 9. Since the initial crack size of this specimen is not known, a definite conclusion cannot be drawn from this figure, but it is very likely that the life prediction based on the crack growth rate model is conservative.

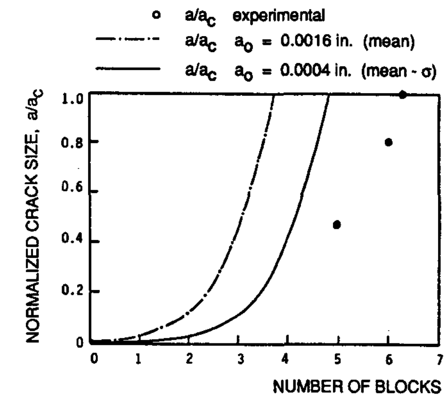


Fig. 9 - Crack growth predictions and experimental results

It should be cautioned that the test specimens used in this study were very small (W4x13) and the numerical values given here should not be extrapolated to large beam-to-column connections. However, the size of test specimens was not deemed to be critical in this study as the purpose of the test series was to investigate a conceptual damage model rather than derive specific numerical values.

DAMAGE MODEL FOR LOCAL BUCKLING

Ten identical cantilever specimens were tested to study deterioration due to local flange buckling in plastic hinge regions in beams. The specimens were similar to those shown in Fig. 3, but using a W6x9 beam section for which the width/thickness ratio of the flanges is large (b/t = 18.9) and local buckling is expected to occur at small inelastic deformations.

A typical load-deflection result of a constant amplitude test is shown in Fig. 2(b). Local buckling did occur early and peak load as well as elastic stiffness deteriorated gradually and in a manner that can be represented closely by the straight-line diagram shown in Fig. 10. The deterioration threshold range (D.T.R.) identifies the number of cycles needed to cause local buckling. This range is followed by a range of relatively fast deterioration (D.R. I) in which the local buckles increase in size almost linearly, and a range of slow deterioration (D.R. II) in which the buckle sizes stabilize. The last range of rapid deterioration (D.R. III) is governed by crack propagation and fracture at the buckles or the beam flange weld. A diagram that illustrates the growth of a buckle is shown in Fig. 11.

The slopes of the lines shown in Fig. 10 represent the deterioration rates per cycle. If these slopes are plotted on a log-log scale against the plastic rotation ranges $\Delta\theta_p$ for all constant amplitude tests, the results shown in Fig. 12 are obtained. A straight correlation line fits reasonably well to the data for deterioration range I (solid line) but less satisfactorily to the data for deterioration range II (dashed line). Thus, it can be postulated that, at least in range I, strength deterioration per cycle, Δd, and plastic rotation range, $\Delta\theta_p$, can be related by an expression of the type

$$\Delta d = A(\Delta\theta_p)^a \tag{7}$$

in which A and a are structural performance parameters.

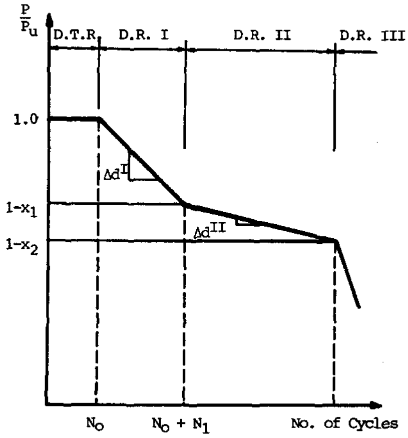


Fig. 10 - Deterioration of peak load

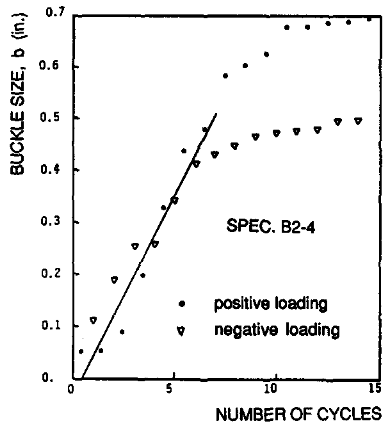


Fig. 11 - Buckle size vs number of cycles

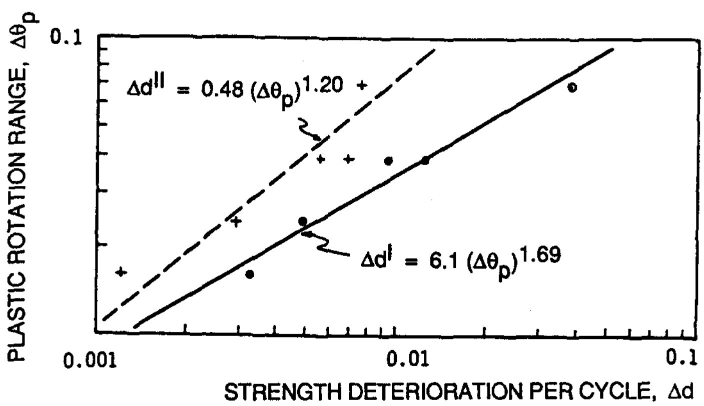


Fig. 12 - Deterioration rate models

If we denote with "x" the limit value of acceptable deterioration that constitutes failure, the number of cycles N_1 spent in the deterioration range can be calculated as

$$N_1 = x/\Delta d = xA^{-1}(\Delta\theta_p)^{-a} \tag{8}$$

This expression is again of the form given by Eq. 1. Presuming that also the deterioration threshold range can be modeled by a similar expression, the number of constant amplitude cycles to failure is given by

$$N_f = N_o + N_1 = C^{-1}(\Delta\theta_p)^{-c} + xA^{-1}(\Delta\theta_p)^{-a} \tag{9}$$

where N_o is the number of cycles spent in the deterioration threshold range.

In order to utilize a damage model of the type given by Eq. 2 for variable amplitude cycling, one of two approaches could be used. One is to use, in sequence, two separate models for the deterioration threshold range and the deterioration range. This approach is difficult to implement because the time sequence of cycles in a loading history will be rearranged during rain-flow cycle counting. The other approach is to use an average deterioration rate for both ranges. In this case the damage model would take the form

$$D = \sum_{i=1}^{N} \frac{1}{C^{-1}(\Delta\theta_{pi})^{-c} + xA^{-1}(\Delta\theta_{pi})^{-a}} \tag{10}$$

This model is only approximate but does not necessitate the use of two sequential damage models. In this formulation the deterioration threshold is ignored and deterioration is assumed to commence at the first excursion but occurs at a rate $\Delta d'$ which is smaller than Δd^I (see Fig. 13).

The accuracy of predicting deterioration with the equation $d = A\Sigma(\Delta\theta_{pi})^a$ (including the deterioration threshold) was examined on a variable amplitude test in which a specimen was subjected to three blocks of a computed seismic response history. The results of the test and the predictions are shown in Fig. 14. The predictions are conservative but quite accurate, particularly when the second deterioration range is included in the model.

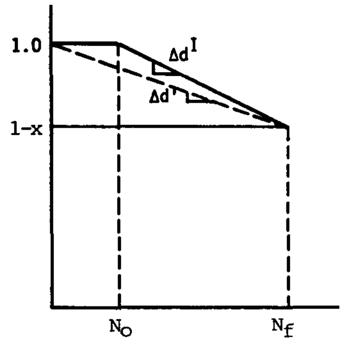


Fig. 13 - Average rate of deterioration

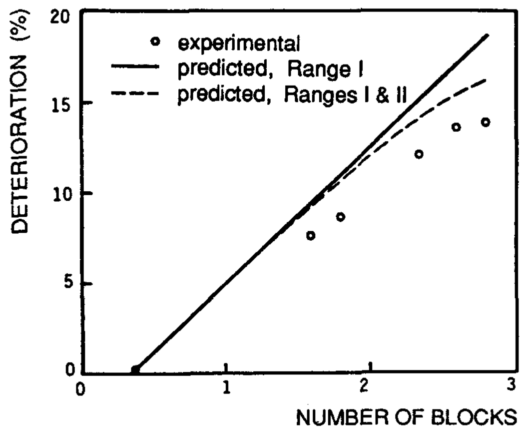


Fig. 14 - Experimental and predicted deterioration

For the two modes of crack propagation and local buckling the postulated model has given reasonable predictions of deterioration and failure. Clearly, the predictions are not exact since the model parameters exhibit scatter, and sequence effects as well as mean deformation effects are ignored. If these limitations can be accepted, the model provides a relatively simple means of predicting component performance for any deformation history the component may be subjected to in an earthquake.

CYCLIC DEFORMATION DEMANDS FOR PERFORMANCE ASSESSMENT

A seismic performance assessment based on cumulative damage models requires information on the cyclic deformation demands imposed by an earthquake on a structure or structural component. According to Eqs. 2 and 3 these demands cannot be described by a single parameter such as a ductility ratio. Information is needed on the number, N, and plastic deformation ranges, $\Delta\delta_{pi}$, of all inelastic cycles.

A comprehensive statistical evaluation of these parameters is in progress for bilinear and stiffness degrading single degree of freedom systems. Twenty California ground motion records are used for this purpose with the records scaled to a common severity level. The scaled records are being used to perform time history analysis on SDOF systems of different yield levels and strain hardening ratios. The response data are being evaluated statistically to obtain a comprehensive set of data on N and $\Delta\delta_{pi}$.

This study is still in progress and final data cannot be reported yet. However, the following general conclusions are of relevance in the context of performance assessment.

1. For a given structural system the plastic deformation ranges, once reordered according to the rain-flow cycle counting method, fit satisfactorily to a lognormal distribution.

2. The frequency of large plastic deformation ranges is very low whereas the frequency of small plastic deformation ranges is very high. Consequently, small inelastic cycles, that individually have very little effect on damage, do affect damage considerably as they occur in great numbers.

3. For a given yield level and period, the number of inelastic excursions increases approximately linearly with the strong motion duration and is only weakly dependent on the strain hardening ratio.

4. The damage predicted by Eq. 2 is much higher for short period structures than for long period structures as the number of inelastic excursions depends strongly on the natural period.

5. For short period structures damage depends stronly on the strong motion duration.

6. For a given structural system, the variations in frequency content of ground motions will lead to damage predictions of considerable uncertainty even if the structural performance parameters C and c are assumed to be deterministic.

Figure 15 shows typical examples of cumulative distribution functions for plastic deformation ranges. The graphs apply for bilinear systems with a period of 0.5 seconds, 5 percent damping, and a yield level of approximately one quarter of the expected elastic force attracted in the system. The graphs show also the effect of the strain hardening ratio α on the distribution of $\Delta\delta_{pi}$. This effect was found to be not very large. In general, the amount of strain hardening was found to be one of the less important parameters in damage evaluation. However, the presence of some strain hardening is important. Elastic - perfectly plastic systems ($\alpha = 0.0$) behaved differently from strain hardening systems as they exhibited considerable drift in the seismic response.

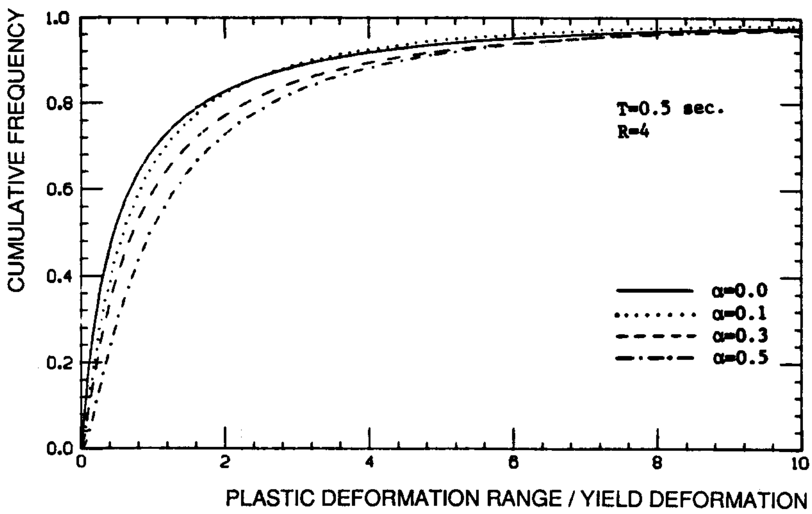


Fig. 15 - Examples of lognormal distributions of $\Delta\delta_{pi}/\delta_y$

Damage, however, is very sensitive to the maximum plastic deformation range, i.e., the largest seismic excursion. This excursion should be used to truncate the lognormal distribution of all other plastic deformation ranges. From the statistical study of the response to 20 California earthquake records it is evident that this maximum plastic deformation range has a considerable scatter and therefore the expression $\Sigma\Delta\delta_{pi}$ has a large scatter. If these uncertainties are added to the uncertainties in the structural performance parameters (primarily in C), it is evident that a realistic damage evaluation must be of probabilistic nature. The data on N and $\Delta\delta_{pi}$ for such an evaluation are presently being developed; the statistical data needed on the structural performance parameters C and c require much more experimental research.

EXPERIMENTAL RESEARCH NEEDS FOR PERFORMANCE ASSESSMENT

The two pilot testing programs discussed previously have indicated that cumulative damage in components of steel structures can in many cases be described by a two-parameter damage model. If this hypothesis can be accepted, experimental research on deteriorating components should be directed towards a determination of the two performance parameters C and c. There is only one type of test that will provide this information, and that is the constant amplitude test. At least two tests with different deformation amplitudes are required to determine the two parameters, and two tests will only suffice if both parameters C and c are nearly deterministic which is rarely the case. If only two tests are performed, the two selected deformation amplitudes should be quite different so that the deduced N_f vs $\Delta\delta_p$ line covers the range of deformation of interest.

Because of the uncertainties in the model parameters, an ideal testing program for performance evaluation will have to consist of a sufficient number of constant amplitude tests to determine C and c from a regression analysis and to obtain measures of dispersion on these parameters. This will require initially a large number of tests, but as more data become available it is conceivable that parts of the problem can be solved through analytical modeling. For instance, in the crack propagation and fracture problem a statistical study on the initial crack size could define most of the uncertainties, and fracture mechanics concepts could lead to analytical solutions for structural performance parameters.

Unfortunately, financial constraints will put a strong damper on the ambitious testing program advocated here. It is anticipated, therefore, that we will continue to do primarily single specimen testing of the type we have done in the past. Although this testing will not provide an answer to the question "how ductile is ductile enough," it will permit a relative performance assessment if we all, worldwide, decide to apply the same deformation history to our test specimens. If we don't do this we will hardly ever be able to use each others test results in a meaningful manner. The author believes that the texact nature of the history selected for this purpose is not of utmost importance as a "best" history does not exist anyway. Maybe some consideration should be given to high-low and low-high sequences in order to incorporate sequence effects in the history.

If we continue to do single specimen testing with different loading histories, it would be useful to base our histories on statistical information on deformation demands rather than on subjective decisions. In this manner we perform at least a proof test. Such statistical information is the subject of the study discussed in the previous section. This information may include a probabilistic distribution of the plastic deformation ranges and statistically acceptable values (e.g., mean or mean+σ) for the number of inelastic excursions, the maximum plastic deformation range, and the mean plastic deformation.

As an example of a loading history generation, let us consider the SDOF system whose cumulative distribution function of plastic deformation ranges is shown in solid lines in Fig. 15. Let us further assume that the response characteristics of the SDOF system are representative of the deformation demands imposed on the component to be tested. For this system with a period of 0.5 seconds a pilot statistical study (Krawinkler et al., 1983) gave the following information: mean number of inelastic excursions is 27, mean plus standard deviation of the maximum plastic deformation range is $6.06\delta_y$, and mean plastic deformation of all inelastic excursions is $1.73\delta_y$ (δ_y = yield deformation). The mathematical description of the CDF shown in Fig. 15 together with the information on the maximum plastic deformation range is sufficient to generate 27 plastic deformation ranges. Loading histories can now be developed whose plastic deformation ranges are equal to the 27 generated ranges once the rain-flow cycle counting method is applied. To obtain these histories, a reverse process of the rain-flow cycle counting method has to be applied since the 27 ranges cannot be linked together directly to form a history.

Two examples of such loading histories are shown in Fig. 16. Although the two histories appear to be totally different, they have identical plastic deformation ranges (shown in the line diagrams to the right of the histories) and an identical mean plastic deformation of $1.73\delta_y$. Thus, both histories are expected to cause the same damage (provided sequence effects are negligible), and both histories contain statistically obtained characteristics of the seismic response of the system.

This approach for selecting seismic loading histories eliminates the bias introduced by selecting histories on a subjective basis. The same approach could be used also to develop a single history, based on statistical seismic demand, that could be employed by all of us for coordinated cyclic testing.

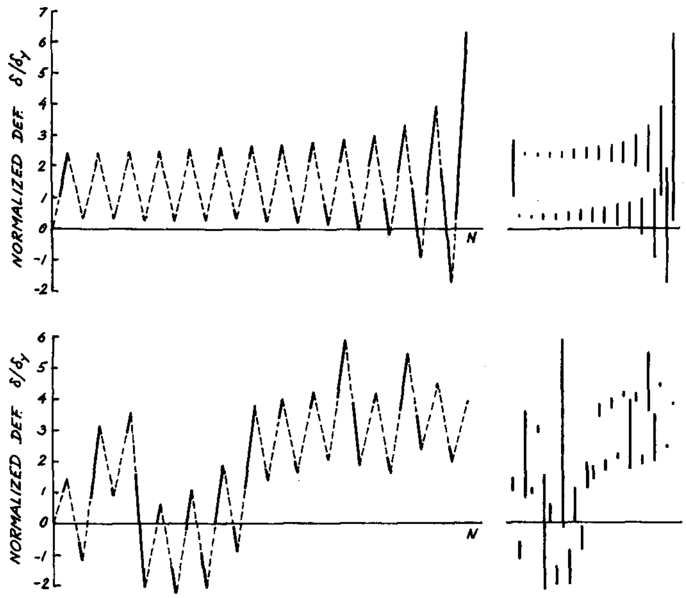


Fig. 16 - Two loading histories with equal plastic deformation ranges

CONCLUSIONS

The study summarized here is a pilot study that has shown certain consistent trends. These trends need to be verified through further reseach. Conclusions based on these trends are summarized below, but it must be understood that these conclusions are preliminary.

1. Various deterioration and failure modes in components of steel structures can be evaluated approximately by means of a simple two-parameter cumulative damage model.

2. This damage model contains two structural performance parameters, C and c. The coefficient C depends strongly on the failure mode and on detailing and exhibits considerable scatter. The exponent c is a much more stable parameter and appears to be usually in the range of 1.5 to 2.0.

3. The parameters C and c can be determined from a testing program that consists of a series of constant deformation amplitude tests. Single specimen testing cannot provide the information needed for a generalization of test results to different seismic response histories.

4. Damage accumulation is a function of the number of inelastic excursions and of the magnitudes of all plastic deformation ranges a component will experience in an earthquake. The number of inelastic excursions, N, increases almost linearly with the strong motion duration of an earthquake. The plastic deformation ranges can be described by lognormal distributions that are truncated with respect to the maximum plastic deformation range. Because of the linear dependence of N on duration, cumulative damage will increase with strong motion duration.

ACKNOWLEDGEMENTS

The research reported in this paper was supported by the National Science Foundation through the grant CEE-7902616. The graduate students M. Zohrei, B. Lashkari-Irvani, and H. Hadidi-Tamjed participated skillfully and diligently in this research.

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