7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Damage Assessment in Steel Structures

M.L. Daali[1]

ABSTRACT

This paper reports on a recently developed damage index model for steel beams under cyclic loading. The model is calibrated using a number of experimental tests and is used to investigate damage in steel structures when subjected to strong ground motion. The main emphasis is placed on assessing actual design rules and associated design philosophy used by standards for ductile moment resisting steel frames.

INTRODUCTION

The design of earthquake-resistant structures relies on the ability of the engineer to adequately balance the demand in energy of the earthquake and the corresponding supply of strength, stiffness, structural ductility and member ductility of the structure. To accomplish the above, it has been accepted practice to satisfy: a) Serviceability conditions, and b) Energy demands by providing members with the ability to absorb and dissipate energy.

Under severe ground motion, structural elements may sustain a high number of inelastic excursions. If sufficiently large, these can cause significant damage in the form of low cycle fatigue magnified by local buckling or cracking. Low cycle fatigue damage has traditionally been of academical interest only. The Northridge (California) earthquake with a 6.8 Richter intensity proved to the contrary that low cycle fatigue damage is indeed a problem that researchers should deal with in practice. After an initial period where only a handful of structural damage assessments have been reported, over sixty steel moment resisting frames built in the mid to late 1980s were later found to be seriously damaged (Degenkolb, 1994).

The advent of recent unsatisfactory building performance suggests some very relevant questions that form the basis of this paper. The questions are as follows: Is the duration of a strong earthquake motion of relevance in the performance of a moment resisting steel frame? Is the nature of the earthquake relevant? Is this performance similar to that expected by building codes? This paper addresses some of these questions by reviewing a recently developed approach for the prediction of damage assessment in steel beams, and finally, applying the latter in a non-linear dynamic analysis of a steel moment resisting frame.

DAMAGE MODELS IN THE LITERATURE

Damage in a structural member or a structure as a whole has usually been predicted in terms of ductility demand parameters. These parameters may depend on displacement, rotation, stiffness or even

[1]Structural Engineer, Hatch Associates Ltd., 128 Pine St., # 103, Sudbury, Ont, P3C 1X3

dissipated energy limits. While some parameters are applicable to a section level, others are used at the element level and others still are used at the overall structural level. Fig. 1 illustrates two of these damage indicators; the first belongs to the normalized energy class, and the second to the ductility class. Comprehensive surveys on the subject of damage indicators may be found in Oliveira (1975), Banon and Veneziano (1982), Powell and Allahabadi (1988), Cheng et al. (1979) and Meskouris and Kratzig (1990).

Most of the damage indicators mentioned above are deficient in one way or another. The class of ductility damage indicators lies on the unconservative side because the repeated cyclic effects are not accounted for. On the other hand, the class of normalized dissipated energy damage indicators is overly conservative because damage accumulated under numerous small amplitude deformation is attributed the same importance as damage under large amplitude deformation. For these reasons, a number of advanced mathematical models for damage assessment have recently been proposed in the literature. Some involve a combination of normalized maximum displacements and normalized dissipated energy. For instance, Park and Ang (1985) proposed a damage index for reinforced concrete members that is defined as a linear combination of the normalized maximum deformation and the normalized dissipated energy. Still other models are based on cumulative damage theory in the form of low-cycle fatigue relationships (Krawinkler and Zohrei, 1983, and Daali and Korol, 1994).

A MODEL FOR DAMAGE ASSESSMENT IN STEEL BEAMS

Damage models that account for maximum deformation and repeated load or low-cycle fatigue effects have been applied recently to practical applications by a number of researchers. For instance, Fajfar and Fischinger (1990) and Fajfar (1992) used this concept to develop modified spectra and equivalent global system ductilities that account for cumulative damage effects in the form of low-cycle fatigue.

It is preferable to limit the number of parameters that can be used to assess damage in a member or a structure to only those effectively informative. Moreover, the damage parameters that should be used must be those response quantities that are easy to obtain or are directly obtainable from results of a structural nonlinear dynamic analysis.

Before formulating any new damage model, it is important to define in a clear and precise manner the term "failure". For ductile steel structures, strength and ductility are the two characteristics that permit the redistribution of forces in the course of an earthquake event. It is, therefore, logical only to define failure of a class 1 (or plastic design) section at the point where it is no longer able to reach its plastic strength. At this stage, damage in the member is associated with a damage index value ranging from zero to one. In this case, zero would mean no damage has occurred in the member, i.e., member has been loaded elastically or slightly beyond (without suffering any form of deterioration). A value of unity would mean that the member is fully damaged and can no longer sustain the plastic limit force anymore.

From the above, it is clear that any damage model should, as a minimum, combine the maximum response with repeated cyclic load effects. As such, the simplest damage model that seemingly can be used is of the form $D=D1+\beta D2$, where D1 is the damage under maximum response, β is a calibration factor and D2 is the damage under repeated cyclic effects.

For the approach to be proposed, consider Park and Ang's (1985) damage model. The expression for damage is given by:

$$D=\frac{\mu_{max}}{\mu_{mono}}+\frac{\beta}{F_p\delta_p\mu_{mono}}\int dE \qquad \textbf{(1)}$$

where D is a Damage Index, F_p is the plastic limit load, dE is the increment of dissipated energy and β is a calibration factor that can be expressed in terms of section and member characteristics. μ_{max} is the maximum experienced ductility and μ_{mono} is the maximum available ductility under monotonic loading.

For research purposes, experimental results are often given in terms of hysteresis loops exhibiting the behaviour of the specimens. Thus, it is difficult to integrate the area under the curves in order to obtain the total dissipated energy. It is, therefore, clear that Eqn. 1 is of limited use because of the difficulty associated with the energy integration term. The aim of this section is to try to simplify Park and Ang's (1985) damage model to yield a simpler expression. As such, let $\sum e_e$, the sum of the normalized energy under every reversal of loading defined by the ratio of the total dissipated energy to the elastic energy be given by:

$$\sum e_e=\frac{\int dE}{(\frac{1}{2})F_p\delta_p}$$ which simplifies Eqn. (1) to

$$D=\frac{\mu_{max}}{\mu_{mono}}+\beta\frac{\sum e_e}{2\mu_{mono}} \qquad \textbf{(2)}$$

To be of value in practice, it is particularly desirable to replace the numerator of the second term in Eqn. 2 by a factor based on cyclic test results. In fact, tests done by Popov and Stephen (1970) and Korol and Daali (1994) on members subject to cyclic loading showed that a linear relationship, of the form $e_e=2.0\pi_d$, exists between the normalized energy, e_e, and π_d, the latter defined as the ratio of the residual plastic deformation to the plastic deformation.

So long as initiation of instability is not significant, one can show that the linear relationship existing between the normalized energy, e_e, and the plasticity ratio, π_d, is also applicable to the cumulated normalized energy, $\sum e_e$, and the cumulated plasticity ratio, $\sum \pi_d$. This suggests that at any time during the loading history, the cumulated normalized energy is linearly related to the cumulated plasticity ratio. Experimental results of tests by Popov and Pinkney (1969), Popov and Bertero (1973) and Korol and Daali (1994) have shown that the cumulated normalized energy is approximately related to the cumulated plasticity ratio by:

$$\sum e_e=2\sum \pi_d \qquad \textbf{(3)}$$

It can be shown that a member's inelastic ductility demand at a point in time is related to the plasticity ratio as:

$$\mu_i=\frac{\Delta_{rp}}{\Delta_p}+1=\pi_d+1 \qquad \textbf{(4)}$$

where Δ_{rp} is the residual plastic deformation and Δ_p is the plastic deformation.
Combining Eqns. 2, 3 and 4, gives:

$$D=\frac{\mu_{max}}{\mu_{mono}}+\beta_2\frac{\Sigma(\mu_i-1)}{\mu_{mono}} \qquad \textbf{(5)}$$

where the term $\Sigma(\mu_i\text{-}1)$ may be interpreted as the cumulated positive and negative residual plastic deformation the member has undergone. An example of the elasto-plastic type response for four load reversals is given in Fig. 2.

Determination of the monotonic ductility μ_{mono}: For the purpose of predicting monotonic ductility, an empirical expression developed by Daali and Korol (1994) is used. The expression is given by:

$$\mu_{mono}=1-9.2\alpha_e+10.71\alpha_e^{-.293} \qquad \textbf{(6)}$$

where α_e is the effective slenderness of the member defined by:

$$\alpha_e=\frac{\alpha_f\alpha_w\alpha_l}{30072} \text{ in which, } \alpha_l=\frac{L}{r_y\zeta}, \quad \alpha_f=\frac{b}{2t\zeta}, \quad \alpha_w=\frac{h}{w\zeta}, \quad \text{and } \zeta=\sqrt{\frac{300}{\sigma_y}}$$

and L, r_y, b, t, h, w and σ_y are the section and the material characteristics expressed in SI units.

Determination of the calibration parameter: For the purpose of determining the calibration factor, β, one can use the definition of failure advanced earlier requiring the damage index, D be equal to unity. Using the damage model of Eqns. 5 and 6, values for the calibration factor can be obtained. For this purpose, cyclic loading data from the results of four tests by Korol and Daali (1994), three tests by Korol et al. (1990) and eleven tests by Castiglioni and Di Palma (1989) were used. The least squares method was used to obtain the expression that fits the experimentally obtained values of the calibration factor as:

$$\beta=0.085(\alpha_f/8.37)^{2.26} \qquad \textbf{(7)}$$

In what follows, we shall demonstrate the use of the above damage model in a steel moment resisting frame.

NON-LINEAR DYNAMIC ANALYSIS OF A SEVEN-STOREY BUILDING

Sample building & computer modeling : The building designed for Victoria, B.C., has moment resisting perimeter frames in one direction and braced frames in the other. Fig. 3 shows a plan view of the sample structure.

A general purpose computer program, DRAIN-2DX, developed by Prakash and Powell (1992) was used for the non-linear dynamic analysis. The sample building was analyzed using beam-column elements with bi-linear moment-curvature and moment-rotation relationships and yield interaction surfaces specified at the ends of the beams and beam-columns. Besides having the ability to calculate the usual member end forces, storey shears at each time step, displacements and rotations envelopes, the program also calculates mode shapes, frequencies, cumulated inelastic rotations, and cumulated energy quantities. The latter are of relevance here since they can be directly used in the damage assessment models developed in the above.

Earthquake records : In general, earthquake records have a large number of parameters that may influence their shape and contents, and subsequently, lead to a difference in structural response of buildings. These parameters range from the location of earthquake to the nature and conditions of soil, the frequency content, the duration of strong shaking and numerous other factors. One major factor among all these can, however, be singled out since it seems to have a significant role on the structural response of buildings. This is termed the frequency content of the record and is commonly referred to as the ratio of peak ground acceleration to peak ground velocity, A/V. It is customary to associate a high A/V ratio with moderate and strong close-by earthquakes while a low A/V ratio is associated with large and distant earthquakes (Naumoski et al. 1993).

For the purpose of the computer analysis, three earthquake records scaled to the same ground acceleration, of 0.3 g, were used. The first one is the TRANS component of the Central Honshu Japan 1971 earthquake and has a very high A/V of 2.56. The second is the N21E component of the Kern County California 1952 earthquake (Taft Lincoln School Tunnel) and has an intermediate A/V of 0.99; while, the last has a very low A/V of 0.52 and is from the N61W component of the San Fernando California 1971 earthquake (2500 Wilshire Blvd, L.A.). The spectral acceleration of the three records are shown in Fig. 4.

Dynamic analysis & damage assessment : Figs. 5 and 6 show the envelopes of maximum lateral deflections and drift ratios. From Fig. 5, it can be clearly seen that the structure behaved adequately under the Central Honshu and the Kern County earthquakes. Under the San Fernando earthquake the structure underwent lateral deflections of the order of 10 to 20% more than the NBCC (1990) limit for lateral inelastic deflections. Similarly, while the Central Honshu and the Kern County earthquakes induced story drifts that were acceptable, those from the San Fernando earthquake are seen in Fig. 6 to exceed the 2% story drift ratio allowed by NBCC (1990) under inelastic behaviour.

Ductility has traditionally been used to measure the level of performance developed by structures. Fig. 7 shows the rotation ductility demands imposed by the three earthquakes on the beams; for instance, the San Fernando earthquake is seen to impose higher ductility demands than the other two. While ductility is seen to permit the comparison between the behaviour exhibited under every earthquake, it will not, however, give the level of damage suffered by the members. The objective of the following is, therefore, to make use of the damage assessment model given by Eq. 5. A summary of the application on the damage index is shown in Fig. 8. Under the Central Honshu earthquake, the structure did not undergo inelastic excursions and as a result, no damage was exhibited. The San Fernando and Kern County earthquakes did however, induce damage in the beams. A study of Fig. 8 shows that damage is significantly greater under the San Fernando earthquake than the Kern County earthquake. Overall, the structural members developed damage indices less than unity. This means that none of the beams has failed. Note, however, that if the structure is subjected to strong after shocks or is subjected to the same earthquakes a second time, within the life-span of the building, some of the beams would probably fail by virtue of the cumulative property of damage under inelastic excursions.

CONCLUSIONS

From the above, one can draw the following conclusions:

1) The response of a building designed according to code requirements is seen not necessarily to respond to an anticipated earthquake in the same manner as is expected from the code. Drift ratios and the corresponding lateral deflections may be larger than the code specified limits.

2) Damage assessment in structural elements can be modelled as a linear combination of maximum response and either repeated effects in the form of low-cycle fatigue or dissipated energy. An approach for damage quantification has been reviewed in this paper. The approach is based on a modified Park and Ang's damage

model. Despite the randomness of any earthquake load event, results achieved with this methodology may be considered acceptable.
3) The application of the damage index has shown that the duration of a strong motion is a significant parameter that should be accounted for. This may be achieved through an explicit non-linear dynamic analysis that is followed by a damage assessment analysis, or in an other format, may be the prescription of more stringent member slenderness limits.

REFERENCES

Banon H. and Veneziano. D., (1982), "Seismic Safety of Reinforced Concrete Members and Structures", Earthquake Eng. and Structural Dynamics, Vol. 10, 179-193.

Cheng F.Y., Oster K.B. and Kitipitayangkul P., (1979), "Establishment of Ductility Factor Based on Energy Absorption and Evaluation of Present Methods", Proc of 3rd Can. Conf. E. Eng., Vol. I, pp. 719-744.

Daali M.L. and Korol R.M., (1994), "Local buckling rules for rotation capacity", Engineering Journal, American Institute of Steel Construction, 2nd Quarter, Vol. 31, No 2.

Daali M.L. and Korol R.M., (1994), " Damage Assessment in Locally Stiffened Beams under Seismic Type of Loading", Eng. Mech. Symposium of C.S.C.E., Winnipeg.

Degenkolb, (1994), "No one expects a new building to fail", EVENTS, Earthquake Reconnaissance News.

Fajfar P. and Fischinger M., (1990), "A Seismic Design Procedure Including Energy Concept", 9th European Conference on Earthquake Engineering, Vol. 2, Moscow, 312-321.

Fajfar. P., (1992), "Equivalent Ductility Factors, Taking into Account Low-Cycle Fatigue", Earthquake Eng. and Structural Dynamics, Vol. 21, 873-848.

Krawinkler, H. and Zohrei, M. (1983), "Cumulative damage in steel structures subjected to earthquake ground motions", Computers and Structures, Vol. 16, #1-4.

Korol, R.M. and Daali, M.L., (1994), "Rotation Capacity and Energy Absorption Enhancement in Beams under Cyclic Loading", Eng. Mech. Symposium of C.S.C.E., Winnipeg.

Korol R.M. et al., (1990), "Extended End-Plate Connections Under Cyclic Loading: Behaviour and Design", J. of Construct. Steel Research 16, 253-280.

Meskouris, K. and Kratzig, W.B., (1990), "Seismic damage assessment of buildings", Earthquake Resistant Construction and Design, Savidis (Ed.).

Naumoski et al., (1993), "Representative Ensembles of Strong Motion Earthquake Records", McMaster University, EERG Report 93-1.

NBCC, (1990), "National Building Code of Canada. National Research Council of Canada", Associate committee on the National Building Research Code, N.R.C., Ottawa, Canada.

Oliveira C.S., (1975), "Seismic Risk Analysis for Site and Metropolitan Area", Report No. EERC-75-3, University of California, Berkeley, CA.

Park Y.J. and Ang, (1985), "A. Mechanistic Seismic Damage Model for Reinforced Concrete", Journal of Structural Engineering, Vol. 111, No 4.

Powell G. and Allahabadi R., (1988), "Seismic Damage Prediction by Deterministic Methods: Concepts and Procedures", Earthquake Eng. and Structural Dynamics, Vol. 16, 719-734.

Popov E.P. and Stephen R.M., (1970), "Cyclic Loading of Full-Size Steel Connections", Report No. UCB/EERC-70/03, EERC, University of Berkeley, CA.

Popov E.P. and Bertero V.V., (1973), "Cyclic Loading of Steel Beams and Connections", Journal of the Structural Division, ASCE, Vol. 99, No. ST6.

Popov E.P. and Pinkney. B., (1969), "Cyclic yield Reversal in Steel Building Connections", Journal of the Structural Division, ASCE, Vol. 95, No. ST3.

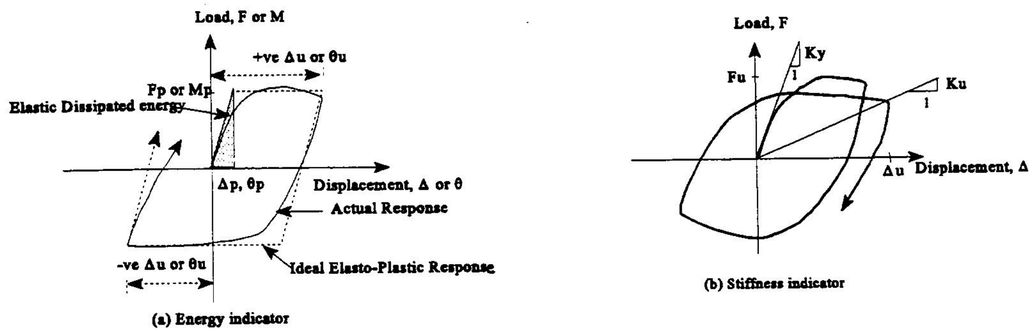
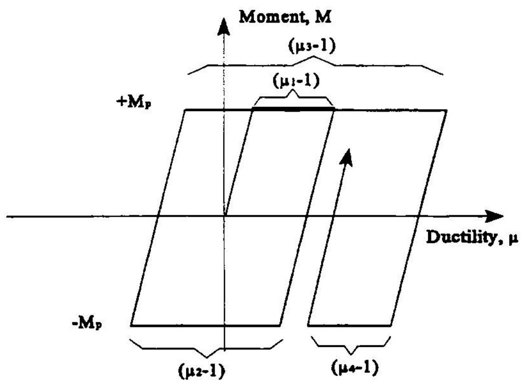


Fig. 1 Damage indicators



$\sum(\mu_i\text{-}1)\text{=}(\mu_1\text{-}1)\text{+}(\mu_2\text{-}1)\text{+}(\mu_3\text{-}1)\text{+}(\mu_3\text{-}1)\text{+}(\mu_4\text{-}1)$

Fig. 2 Cumulated residual plastic deformation

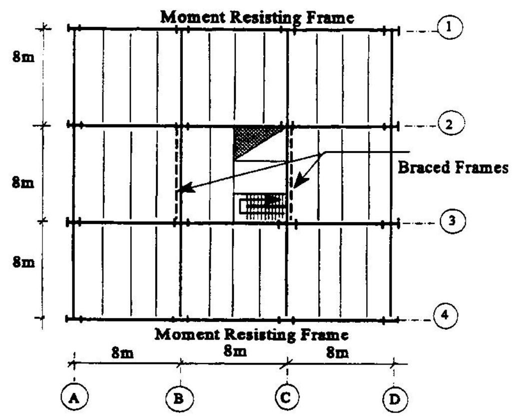


Fig. 3 Plan view of sample building

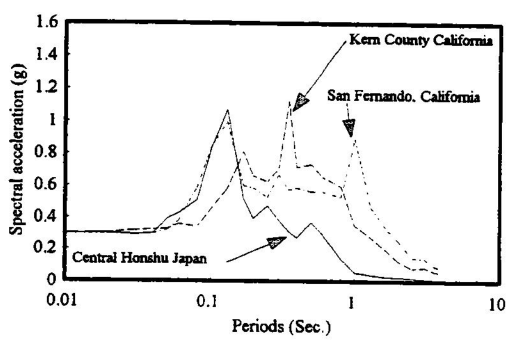


Fig. 4 Earthquake response spectra

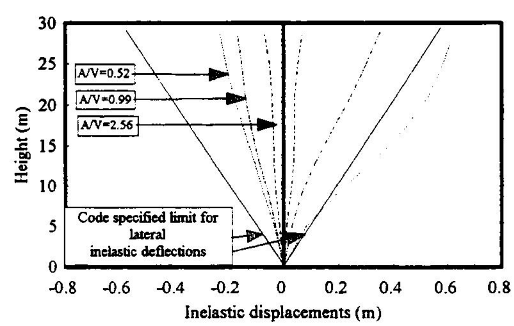


Fig. 5 Envelope of lateral displacements

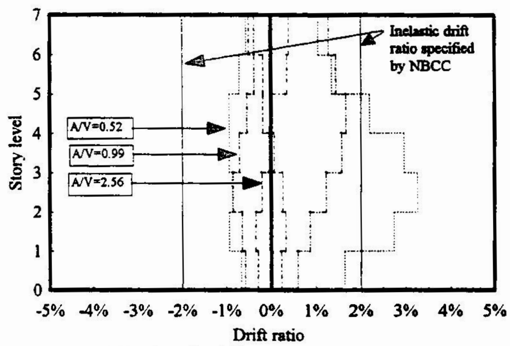


Fig. 6 Envelope for drift ratios and code specified drift limit

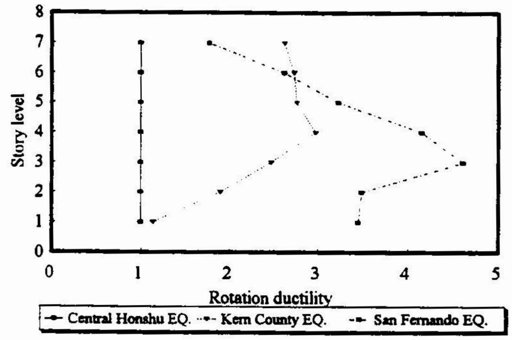
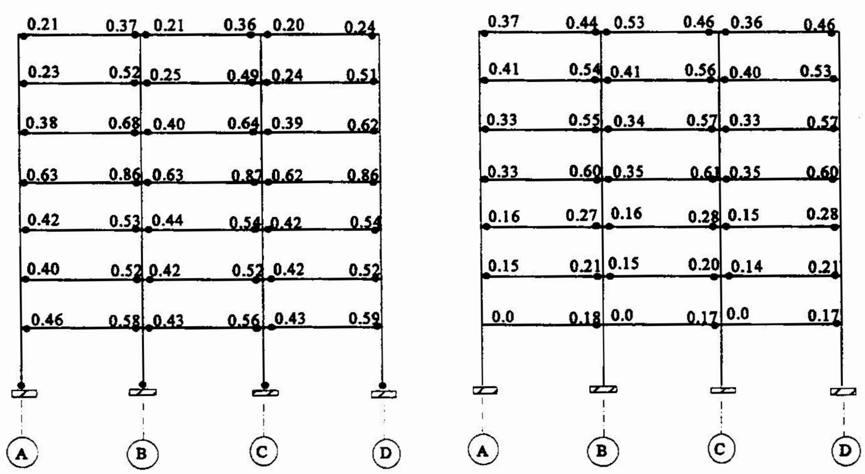


Fig. 7 Earthquake ductility demands



(a) (b)

Fig. 8 Damage assessment under: a) San Fernando California earthquake 1971; b) Kern County California earthquake 1952.