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Seismic Damage Assessment in Concrete Structures

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ABSTRACT

The assessment of seismic risk, and the subsequent decisions regarding retrofit, require that the damage to the structure in the future be predicted in a manner that permits evaluation of consequences. In the case of reinforced concrete structures, a promising way to define damage of the structural system is through the use of damage indices. Of special interest is the prediction of damage in older structures, detailed prior to development of recent seismic design concepts. A number of the indices that have been proposed in the literature were reviewed to compare their characteristics and to determine suitability for quantifying damage states of reinforced concrete structures. Damage indices were computed for a series of combined shear-flexure tests carried out recently at UBC (Adebar et al 1995), and for the Oak Street Bridge bent test program (Anderson et al 1995), providing a means to relate the numerical values of calculated damage index to actual observed damage in large scale models of realistic concrete structures. The evaluation of the various indices is discussed in detail by Williams and Sexsmith (1994). Among the several promising candidate damage indices is the one proposed by Park and Ang (1985).

In the case of the Oak Street Bridge bents, the IDARC computer program (Kunnath et al 1992) was used to predict behaviour of the tested models and corresponding damage index for comparison with observed behaviour. IDARC performs an inelastic analysis, including calculation of the Park and Ang damage index. After some modelling to deal with certain limitations of the program, such as its inability to deal with beam shear, excellent agreement was found between the actual test hysteresis loops (under displacement control) and the analytical model prediction. This agreement was reached for each of several tests involving several different retrofit schemes for the structure. With the response in good agreement with model test results, the calculation was extended to the prototype bridge piers, and damage index was determined for several seismic records. The response of the prototype demonstrated an extreme degree of sensitivity to the levels of peak acceleration in the records, and this raises interesting questions about the relationship between slow cyclic displacement controlled tests, as performed in the laboratory, and the situation in an actual seismic event.

INTRODUCTION

The existing stock of buildings and bridges includes a very large proportion of structures that are deficient by today's seismic standards. In order to manage this inventory of buildings, owners

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have to consider the consequences of taking no action, as well as consequences of embarking on retrofits of varying effectiveness and cost. Principles of decision analysis are appropriate for this situation. For any given prospective course of action, ie a particular retrofit, the immediate construction cost of the retrofit should be added to the expected cost of the consequences discounted to the present. This total, the present expected cost, then represents an equivalent one time lump sum payment associated with the retrofit. For a single structure, the action that minimizes this total is a possible optimal choice, subject to regulatory and legal constraints. For a group of structures, ie all highway bridges in a province, retrofit funds can be prioritized so that the first funds expended can represent the greatest savings in present expected cost per unit of retrofit expenditure (Sexsmith 1994). The consequences are probabilistic in nature, and are a function of the damage, hence the need to be able to predict damage for all possible seismic scenarios.

DAMAGE INDICES

Classification of seismic damage has been primarily concerned with post-seismic situations. For example, ATC (1989) gives guidance on the assessment of buildings, in which structures are assessed as safe or unsafe, based on structural criteria such as crack sizes, spalling, and lean of columns. The typical retrofit decision problem, however, arises prior to a damaging earthquake. In this case, a damage prediction is required, rather than a damage assessment. The prediction is of necessity a probabilistic one, depending on assumptions about the seismicity at the site and the response of the structure to the possible seismic events in the future. Part of this problem is the prediction of damage for a specific seismic input. A number of researchers have proposed damage indices to relate damage to such input. The numerical damage indices proposed in the literature have two major deficiencies. First, only limited calibration has been performed against observed damage in laboratory tests or post-earthquake investigations, restricting the ability to attach physical meaning to the numerical values calculated. Second, the indices are primarily based on an assumed flexural mode of failure. While this is the preferred mode for modern concrete structures, most older structures are limited by a brittle shear failure mechanism. The ability of damage indices to model such mechanisms is largely untested.

In order to improve this situation, eight proposed measures of local damage were examined in a recent study (Williams, Villemure, and Sexsmith 1995). The investigation consisted of a calculation of each of the proposed damage indices for the slow cyclic loading of a series of specimens tested as part of a comprehensive shear investigation by Adebar at UBC (Adebar et al 1995). Adebar's test program included photographs of the specimens at many stages of the loading history. These were made available by Adebar, permitting examination of correlation of visible damage and its severity with the calculation of damage index.

Based on this study of a limited number of test specimens, it appears that none of the indices studied showed obvious shear-dependent trends. Shear damage appeared to be dependent on the deformation level, generally independent of the number of cycles of load. The simple deformation-based indices seemed the most appropriate. These are the ductility ratio

$$\mu = \delta_m/\delta_y \tag{1}$$

where δ_m and δ_y are the maximum and yield displacements, respectively; the modified stiffness ratio of

Roufail and Meyer (1987)

$$D = (k_f/k_m)(k_m - k_0)/(k_f - k_0) \qquad (2)$$

where k_f and k_m are the secant stiffness at failure and at maximum deformation, respectively, and k_0 is the initial tangent stiffness; and the modified Park and Ang index

$$D = (\delta_m - \delta_y)(\delta_f - \delta_y) + \beta\{\int dE\}/F_y\delta_f \qquad (3)$$

where β is a strength degradation parameter, around 0.1 for well-reinforced concrete, δ_f is the displacement at failure, E is the energy in the cycle, and F_y the yield force. Displacements and force may be generalized, ie to rotations and moment (Park and Ang 1985). The Park and Ang index is implemented in the flexural damage calculation of the IDARC computer program (Kunnath et al 1992). In this case the overall damage index for a complete structure is found by taking the weighted average of local indices found from (3), in which the weighting factors are proportional to the energy absorbtion at a given location.

INELASTIC ANALYSIS OF OAK STREET BRIDGE PIERS

The tests of large scale models of the Oak Street Bridge piers (Anderson et al 1995) provide a rare chance to compare the non-linear analysis capabilities of the IDARC program and the related damage index calculations with observed behaviour. Models of the original Oak Street (OSB1) and Queensborough (QB1) bents, and of one of the retrofitted Oak Street bents (OSB2), were analyzed for the same displacement history as was performed on the physical models. Shear damage was represented by making modifications to the element flexural properties (see Williams 1994). The as-built Oak and Queensborough models represent practice from the 1950's for concrete detailing, and are therefore representative of many bridge structures in use today and requiring retrofit.

IDARC (Inelastic Damage Analysis of Reinforced Concrete) was originally developed at the National Earthquake Engineering Research Center in Buffalo, NY. The most recent release is version 3.0 (Kunnath et al 1992). This version, with some modifications, was utilized to model the bridge piers. The details of development of the models and the related hysteresis parameters are discussed in Williams (1994).

Figure 1 shows the test hysteresis of the OSB1 specimen, in terms of total lateral load versus joint displacement. The IDARC analysis for the same displacement history produced the hysteresis loops of Figure 2. The excellent agreement indicates that the calibration of the model has been successful, at least as far as global hysteresis. The Park and Ang damage index is plotted as Figure 3. The analysis steps are equally distributed throughout the analysis cycles; the precise correspondence of analysis steps with cycles can be determined by reference to Williams 1994. The damage index remains low up to displacement cycles of 11mm. At this displacement yielding of the bent has occurred. After this, D increases rapidly, reaching a value of 1.0 at about the onset of the 22mm cycles. At this point, the cap beam had suffered severe cracking just inside the columns, with horizontal cracks along the top reinforcing steel. Thus it would be reasonable to describe this damage as "failure". The development of the damage index therefore seems to correlate well with the level of damage sustained by the bent. The distribution of damage in the computer model also reflected quite accurately the damage distribution in the test bent., where the most severe damage was concentrated in the beam, just inside the columns (see Anderson et al 1995). Figure 3 shows that the damage index is almost entirely due to the contribution from the beams.

The retrofit specimen OSB2 produced the test hysteresis shown in Figure 4, while the IDARC model result is in Figure 5 and the damage index in Figure 6. The good agreement in the hysteresis curves shows that the model accurately reflects behaviour, and lends credence to the damage index results. In this test, the cap beam has been prestressed, thus improving its flexural and shear capacity. The damage index of Figure 6 is now dominated by yielding of the column tops, with very little damage in the beam elements. The damage increases quite steadily through the analysis, and stays below 0.8 until the last cycle. In the test, the beam suffered heavy cracking prior to the last cycle and then suffered a sudden column shear failure, which was not modelled in the IDARC model. The test behaviour up to the last cycle thus agrees well with the damage evolution given by the analysis.

The results for the QB1 test are shown in Figures 7, 8, and 9. The analysis hysteresis pattern is again a good reflection of the test behaviour. In this test the damage index reached 1.0 while the test specimen had some remaining capacity, thus the damage index was rather conservative. Further, the distribution of damage in the analysis was too high in the beams. In the test, damage was heavy in the column tops and the joints, and limited to the ends of the cap beam. Sensitivity studies with the hysteresis parameters in this case showed rather high sensitivity of the damage index to the hysteresis parameters.

SEISMIC ANALYSIS OF THE PROTOTYPE BENT

The three tests and their corresponding analytical models showed reasonably good agreement in the hysteresis loops and in the calculated damage index. This provides some confidence in the results of an analysis for a seismic load history on the corresponding prototype bents. The prototype bents were modelled by scaling up from the properties and dimensions of the models. Several seismic records were used, each scaled to a variety of peak accelerations. The analyses are discussed in detail elsewhere (Williams 1994), and the discussion that follows provides only a brief example of the OSB1 test.

OSB1 prototype bent represents the as-built bent in the actual bridge. Figure 10 shows time history of cap beam acceleration and displacement for bent OSB1 subjected to the YERMO NS record, scaled to a pga of 0.16g, and the corresponding damage index. The large displacement peak at around 17 seconds caused a yielding in element 6 of the cap beam, resulting in a slight offset in the final displacement, and a final damage index of 0.193. The steady increase in damage index beyond the peak acceleration point is due to hysteretic energy absorbtion.

Figure 11 shows the response for the same input excitation, scaled to a pga of 0.17g. The early parts of the response are almost the same as before, but failure now occurs at 15 seconds. Yielding commences in the cap beam and very quickly spreads throughout the structure, resulting in complete failure within a few hundredths of a second.

Many similar analyses have been performed by the first author, with the extreme sensitivity showing up in almost all cases (Williams 1994). It will require further analysis, and comparisons with other models, to fully explain this extreme sensitivity. Meanwhile, it seems reasonable, and consistent with performance in earthquakes, to assume that structures that are not highly redundant will demonstrate such sensitivity. In the case of the Oak bents, one shear release in the cap beam is sufficient to convert the structure to a mechanism, hence to fail. Two hinges have the same effect, and since there is symmetry to the reverse loading, two hinges can form at almost the same time, resulting in a mechanism.

CONCLUSIONS

It is generally accepted that many buildings and bridges constructed prior to the late 1970's do not meet current seismic standards. Recent experience in the 1995 Kobe earthquake, the 1994 Northridge earthquake, the 1989 Loma Prieta earthquake, and many others demonstrates that the new structures perform well and that the older ones are at risk. The enormous cost of upgrading the older structures requires that appropriate decision strategies be developed that can establish the optimum balance between seismic retrofit cost and seismic consequences. Indices of damage are an obvious possible part of the decision process.

While none of the damage indices examined in the recent investigation discussed herein proved to be ideal, the Park and Ang index was among the several that showed reasonable agreement with observations on the Adebar test series, and also correlated well with observed damage in the Oak Street bent tests. As part of the IDARC package, it becomes a useful tool for damage studies in realistic structures. The amount of damage and the location of damage are identified in this type of analysis, thus it can be a useful tool in retrofit studies.

The damage index and related behaviour in the IDARC analysis of the prototype bents demonstrated an extreme sensitivity of the behaviour to the scaled seismic records. Because most testing is by displacement control, it does not demonstrate this sensitivity well. The failures detected in the IDARC analysis are due to the fact that for the two-hinged two column bents of Oak Street, introduction of only one shear failure plane, or of two flexural yield zones, results in a mechanism and thus failure. Such structures, where a small number of localized failure zones results in global failure, are likely to be particularly sensitive to a peak pulse in the seismic input; this may not be obvious from slow cyclic testing.

The damage index concept, subject of much current research, appears to be a useful part of the seismic retrofit decision process. As discussed here, it only deals with structural damage. Other measures would be needed for architectural damage and perhaps collapse risk for life loss estimates.

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FIGURES

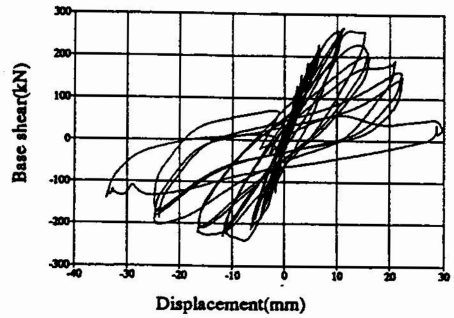

Fig. 1 Test hysteresis for OSB1

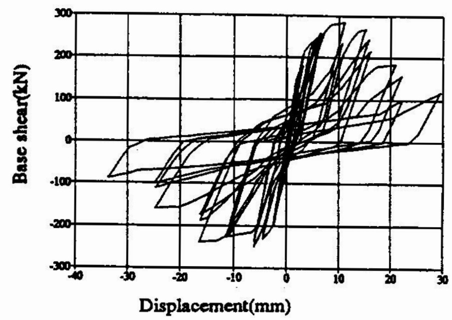

Fig. 2 IDARC model hysteresis for OSB1

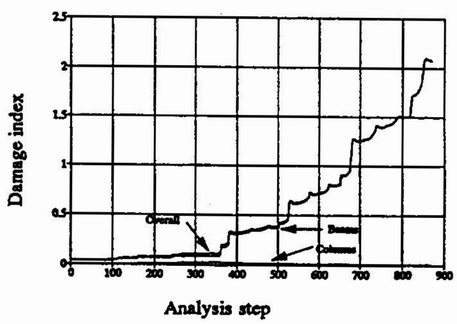

Fig. 3 Damage index for OSB1

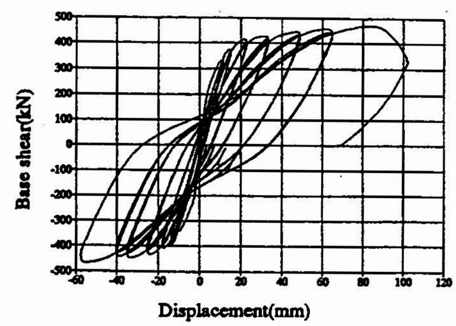

Fig. 4 Test hysteresis for OSB2

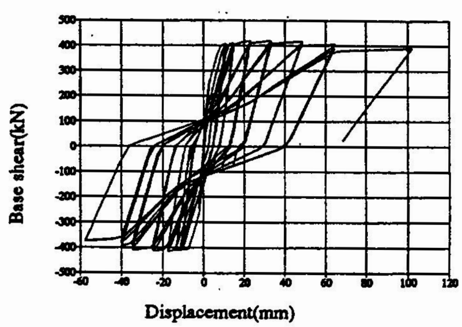

Fig. 5 IDARC model hysteresis for OSB2

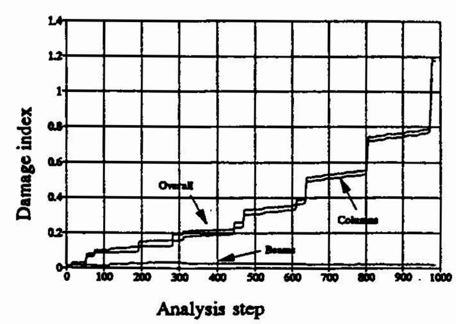

Fig. 6 Damage index for OSB2

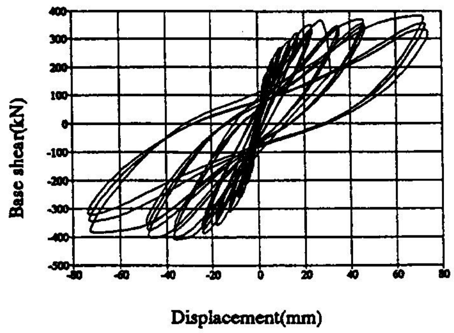

Fig. 7 Test hysteresis for QB1

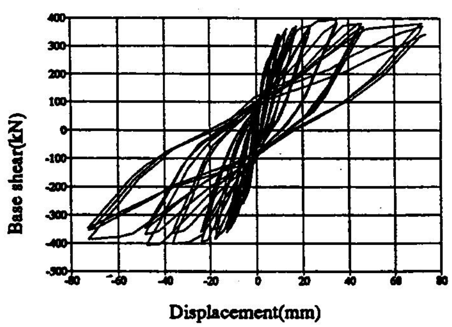

Fig. 8 IDARC model hysteresis for QB1

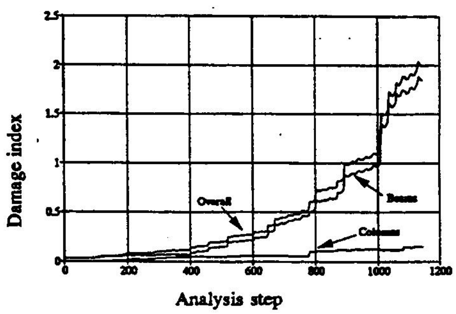

Fig. 9 Damage index for QB1

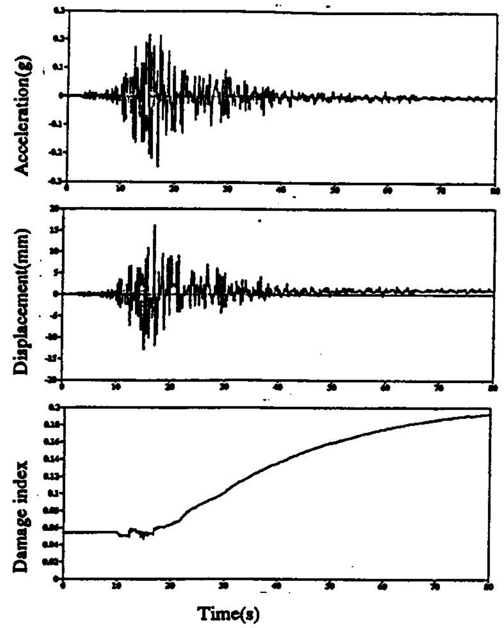

Fig. 10 Prototype Analysis, pga 0.16g

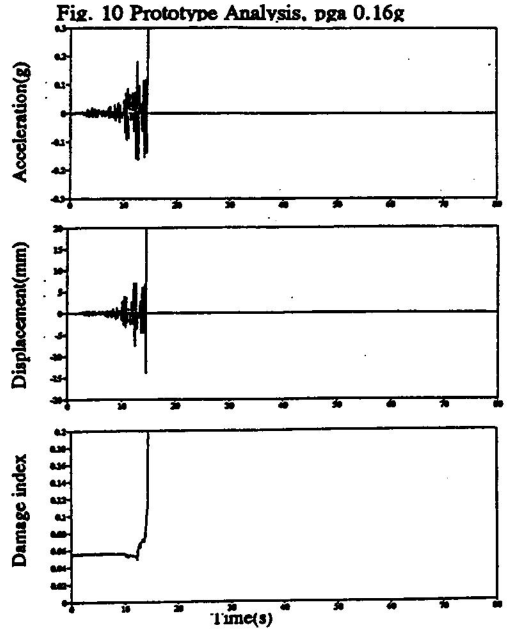

Fig. 11 Prototype analysis, pga 0.17g