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Evaluation of seismic performance parameters

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ABSTRACT: Recent earthquakes have shown that buildings designed for life safety could sustain severe damage to structural and nonstructural elements and building contents. In the U.S., several groups have proposed performance-based seismic design approaches to produce buildings that will perform during earthquakes in conformance with the objective of design, such as immediate occupancy and collapse prevention. In the design process, the response of structural members and systems are checked against pre-established threshold levels of performance which are usually established by laboratory tests. This paper examines the influence of testing protocol on the outcome of experimental work which establishes threshold levels of structural responses to seismic loads.

1. INTRODUCTION

Recent earthquakes have shown that buildings designed primarily for life safety according to the seismic provisions of current building codes could sustain severe damage to structural elements, nonstructural elements, and building contents. Although no loss of life occurred in modern buildings designed to relatively current building codes in recent earthquakes, economic losses from building damage have been staggering. For example, the economic loss from the Northridge earthquake (M7.1) has been estimated at more than $20 billion. This is clearly an unacceptable level of loss for relatively frequent and moderate earthquake. Furthermore, many critical buildings such as hospitals, communication centers and police stations, were not useable immediately after the earthquake due to damage to structural and nonstructural elements. Thus, design professionals recognize that buildings should be designed not only for life safety of occupants but also to control damage.

To accomplish these dual objectives, a new approach to seismic design procedures must be developed. Several efforts are underway in the U.S. to formulate design guidelines which will produce structures of predictable seismic performance. Performance-based seismic design enables the designer to produce buildings that will perform during earthquakes in conformance with the objective of design. In general, four kinds of information are needed for a performance approach to design. (1) Owners or users and their designers first need to set out the requirement or objective of building performance. (2) A precise statement of criteria is then prescribed to indicate the threshold level of performance that must be met to assure that requirements have been satisfied. (3) The evaluation method is then identified. This may include analytical methods, physical testing, or expert judgement. (4) Finally, a commentary is required to assist in clarifying the requirement, criterion or method of evaluation.

In performance-based seismic design, the required levels of performance may be stated as: (1) continued operation, (2) immediate occupancy, (3) life safety, and (4) collapse prevention. A number of documents have incorporated similar objective statements [ICSSC 1994, FEMA 1996, SEAOC 1995]. At each stage of the design process, the response of structural members and systems are checked against threshold levels of performance in terms of response parameters to see whether a stated performance objective is being met. The response parameters can be expressed in terms of stresses, displacements, ductility, energy dissipation, and other criteria.

Modern seismic design relies upon the inelastic response of structural members and systems to dissipate the energy imparted to a structure by an earthquake. One objective of the seismic design process is to limit the locations of damage and to ensure that damage at these locations is within acceptable limits. To facilitate the quantification of damage in analysis and design, it is helpful to define damage in terms of a numerical "damage index." Although many damage index theories have been developed, there is no commonly accepted standard method of numerically defining damage. However, it is known that most damage indices are dependent on the loading history, or "load path" to which a member is subjected. The inability of most

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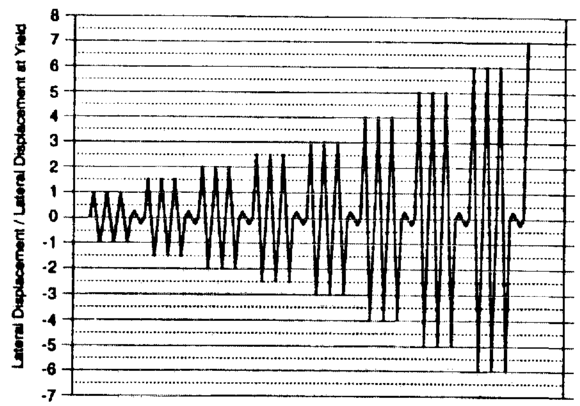

Figure 1: Typical laboratory displacement history

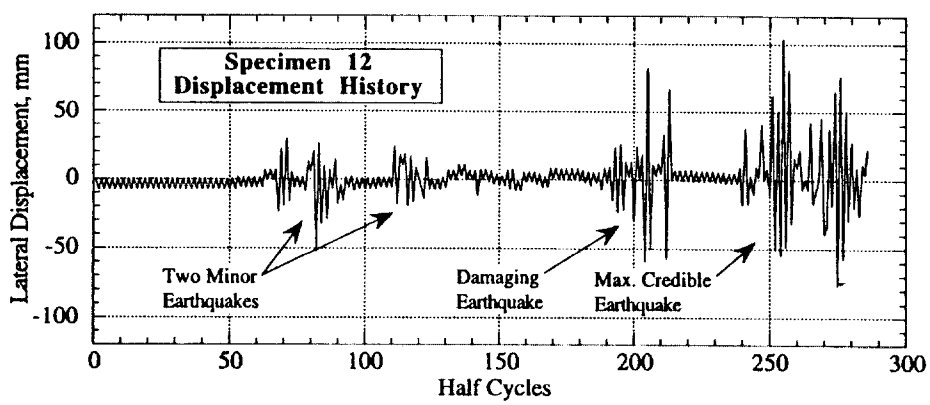

Figure 2: Typical earthquake displacement histories

damage index models to accommodate varying load paths is a major stumbling block in the development of a practical damage index. This paper illustrates the importance of load path in determining the damage exhibited by a particular class of structural members - reinforced concrete columns.

2. INELASTIC RESPONSE OF COLUMNS

To study the relationship between the state of damage and inelastic response of structural member in laboratories around the world, a large number of tests have been performed in which cyclic lateral displacements were applied to the top of cantilever reinforced concrete columns. Although these tests were performed to study the seismic response of the columns, the displacement pattern applied to the columns in most cases did not resemble the displacements induced by an earthquake. The displacement patterns were usually in the form of a sawtooth wave, often with gradually increasing amplitude, as illustrated in Figure 1. However, earthquakes impart displacement patterns with random amplitudes, as shown in Figure 2. An experimental program consisting of 12 nominally identical specimens has been carried out at NIST to show the relationship between regular, sawtooth displacement patterns widely applied in the laboratory (Figure 1), and the random displacement patterns induced by earthquakes (Figure 2). Figure 3 shows a test specimen and the loading apparatus. The specimens had circular cross sections and were nominally identical. The scale of the specimens was 1:4, and the full scale pier on which the design was based conformed to California Department of Transportation (Caltrans) specifications [Caltrans, 1992]. Figure 4 shows the arrangement of reinforcement in the specimen and base block. The column confining reinforcement was a continuous smooth spiral wire, which extended through the depth of the specimen base block. Longitudinal reinforcement consisted of 21 deformed bars distributed around the perimeter of the spiral, extending

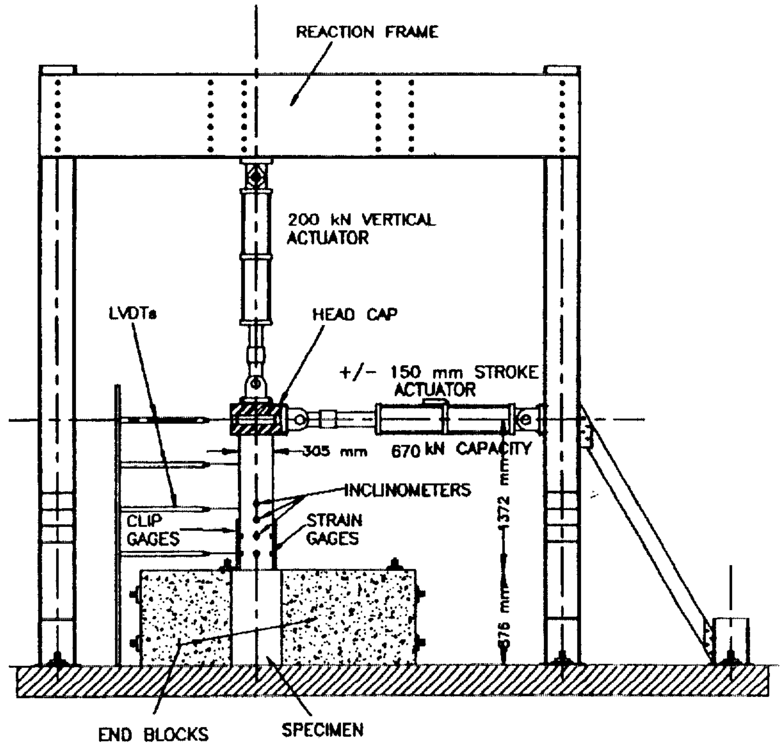

Figure 3: Load frame and instrumentation for cyclic lateral load tests

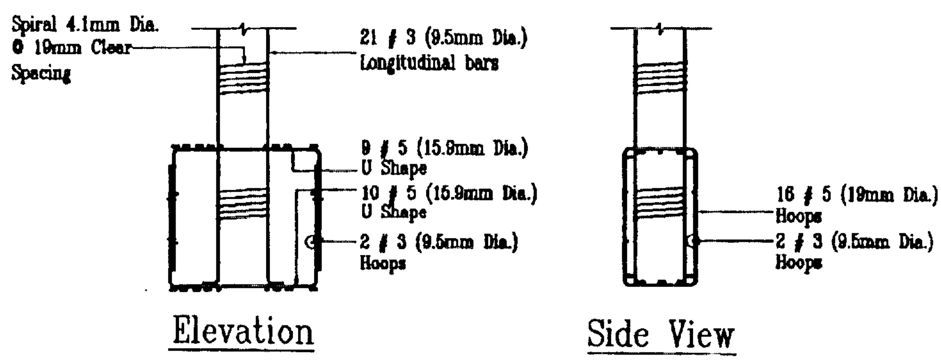

Figure 4: Reinforcement details for column specimen and base block

into the base block and terminating in hooks. The measured strength of the concrete (using 150 mm by 300 mm cylinders) was 38 MPa. The maximum concrete aggregate size was 13 mm. The yield strength of the longitudinal reinforcement was 414 MPa, and the yield strength of the spiral reinforcement was 427 MPa.

The load paths of the first six specimens are illustrated in Figure 5. Specimen 1 was subjected to monotonically increasing lateral displacement to failure. Since all specimens are nominally identical, this test established basic parameters, such as initial lateral stiffness, displacement at first yield, and ultimate lateral strength, which were used in planning subsequent tests. Specimen 2 was subjected to a "standard" laboratory displacement history, detailed in Figure 1. Groups of cycles are applied with increasing amplitudes. This continues until substantial degradation of the lateral load carrying capacity of the column is observed, usually defined as the point at which the maxim lateral load in a cycle is only 75 percent to 80

Table 1: Earthquake records applied to Specimens 7 to 12

Spec. No.	Event No.	Description: Event Severity and Number	Purpose of Applying Event to Column	Earthquake Record	Scale Factor	Scaled PGA g's
7	1	Damaging No. 1	Significant damage	Loma Prieta 1989, Presidio	12.0	1.20
	2	Minor No. 1	Aftershock	Imperial Valley 1979, Superstition Mtn.	1.8	0.34
	3	Minor No. 2	Second aftershock	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	4	Damaging No. 2	Failure of column	San Fernando 1971, 455 S. Figueroa St.	3.6	0.54
8	1	Minor No. 1	Minor damage	Imperial Valley 1979, Superstition Mtn.	1.8	0.34
	2	Minor No. 2	Minor damage	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	3	Damaging No. 1	Significant damage	Loma Prieta 1989, Presidio	12.0	1.20
	4	Damaging No. 2	Failure of column	San Fernando 1971, 455 S. Figueroa St.	3.6	0.54
9	1	Damaging No. 3	Significant damage	San Fernando 1971, Orion Boulevard	3.25	1.43
	2	Minor No. 2	Aftershock	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	3	Damaging No. 4	Moderate damage	El Centro 1940	1.0	0.35
	4	Minor No. 3	Aftershock	San Fernando 1971, 455 S. Figueroa St.	1.0	0.15
	5	Damaging No. 3	Failure of column	San Fernando 1971, Orion Boulevard	3.25	1.43
10	1	Minor No. 2	Minor damage	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	2	Damaging No. 4	Moderate damage	El Centro 1940	1.0	0.35
	3	Minor No. 3	Aftershock	San Fernando 1971, 455 S. Figueroa St.	1.0	0.15
	4	Damaging No. 3	Significant damage	San Fernando 1971, Orion Boulevard	3.25	1.43
	5	Damaging No. 3	Failure of column	San Fernando 1971, Orion Boulevard	3.25	1.43
11	1	Damaging No. 4	Significant damage	Northridge 1994, VA Hospital	1.0	0.42
	2	Minor No. 4	Aftershock	Northridge 1994, Griffith Observatory	1.0	0.26
	3	Minor No. 5	Minor damage	Taft 1952	1.0	0.36
	4	Damaging No. 5	Failure of column	Mexico City 1985, SCT	1.0	0.17
12	1	Minor No. 4	Minor damage	Northridge 1994, Griffith Observatory	1.0	0.26
	2	Minor No. 5	Minor damage	Taft 1952	1.0	0.36
	3	Damaging No. 4	Significant damage	Northridge 1994, VA Hospital	1.0	0.42
	4	Damaging No. 5	Failure of column	Mexico City 1985, SCT	1.0	0.17

percent of the peak lateral load capacity measured during the test. Specimens 3 to 6 were subjected to constant-amplitude cycles ($\pm 2\Delta y, \pm 3\Delta y, \pm 4\Delta y$, and $\pm 5\Delta y$, respectively) until severe deterioration of the column was observed.

A typical displacement history for Specimens 7 to 12 is shown in Figure 2. The details of the displacement histories for these specimens are shown in Table 1. These last six specimens were used to study the effects of real earthquake displacement histories on accumulation of damage, particularly over a series of events of varying intensity. The displacement patterns applied to Specimens 7 to 12 were derived from measured earthquake acceleration records. They were selected from a pool of available strong ground motion records recorded at bedrock sites in the state of California, and one record from Mexico City. Each specimen was subjected to a series of up to five earthquakes. Table 1 shows that earthquakes of a range of intensities were applied, and the order in which they were applied was varied. The pool of available earthquake records was subdivided into two main categories: minor events and major events. Minor events were intended to simulate either isolated low-intensity earthquakes, or low intensity aftershocks following a major earthquake. Minor events would cause little or no damage to the specimens in this test program, as determined from the results of the constant amplitude tests (Specimens 3 to 6). Based on observations from these tests, minor events were defined as those which caused three to five excursions into the inelastic range, with ductility demands no greater than $3\Delta y$. Major events were divided into two types: damaging earthquakes and maximum credible events. Again, based on the results of the earlier constant-amplitude tests, damaging events were defined as those which caused multiple excursions into the inelastic range, with ductility demands greater than $3\Delta y$, but generally less than $5\Delta y$. The earlier tests showed that this level of ductility demand caused deterioration of the column, but that the column retained much of its strength and stiffness. Maximum credible events were those which caused cycles with ductility demands greater than $5\Delta y$. The earlier tests also showed that just a few excursions greater than $5\Delta y$ resulted in rapid deterioration of strength and stiffness. In some cases, where acceleration records were not available which met the desired criteria, the records were modified by simple amplitude scaling. These scale factors are reported in Table 1.

3. DAMAGE OBSERVATIONS

3.1 *Constant Amplitude Tests*

It was observed that repeated cycling of Specimen 3 at a displacement amplitude of $\pm 2\Delta y$ caused almost no degradation of stiffness and strength of the column. After

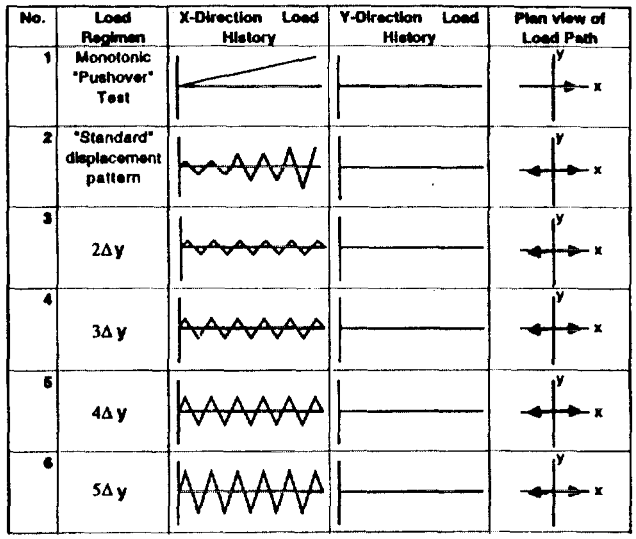

Figure 5: Displacement histories of Specimens 1 to 6

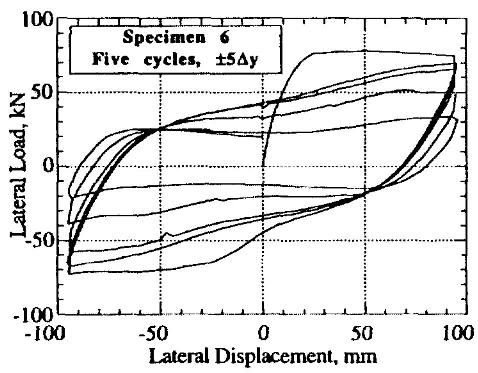

Figure 6: Lateral load-displacement plot, Spec. 6

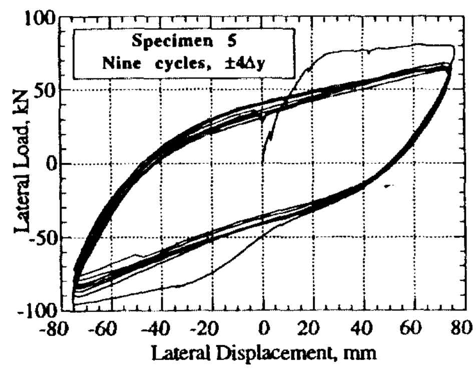

Figure 7: Lateral load-displacement plot, Spec. 5

the first full cycle at $\pm 2\Delta y$ (during which initial yielding and cracking occurred) the hysteresis loops remained extremely stable, lying nearly on top of one another for 150 cycles. The test was stopped at 150 cycles not because the column failed, but because it was believed that no further useful information could be obtained by continuing the test. At the other extreme, Specimen 6 was subjected to cycles of $\pm 5\Delta y$ and exhibited a rapid decrease in strength and stiffness, nearly completely losing lateral stiffness and load capacity after only 5 cycles. This rapid deterioration is illustrated in Figure 6. Specimen 5, which was cycled at $\pm 4\Delta y$, exhibited a gradual decrease in strength and stiffness as cycling progressed (Figure 7), but the decrease was not nearly as rapid as for Specimen 6. This is illustrated in Figure 7.

Thus, for the specimens tested in this program there appears to be a threshold ductility level, of about $4\Delta y$, above which deterioration is rapid and severe. It could be surmised that earthquakes which induce displacements of less than $4\Delta y$ in the columns tested in this program would cause much less damage than those which induce displacements greater than $4\Delta y$. Indeed, subsequent testing under earthquake displacement patterns (Specimens 7 to 12) confirmed this observation: a series of several minor events, which caused few excursions greater than $2\Delta y$, would result in very little damage to a column, while a single earthquake with a few excursions greater than $4\Delta y$ would result in rapid deterioration of strength and stiffness. The apparent threshold of $4\Delta y$ applies only to the columns tested in this program. It could be expected that similarly designed circular columns, with high ratios of confining reinforcement, and which were dominated by flexural rather than shearing deformations, would also exhibit such a threshold, but the level of the threshold might not necessarily be $4\Delta y$.

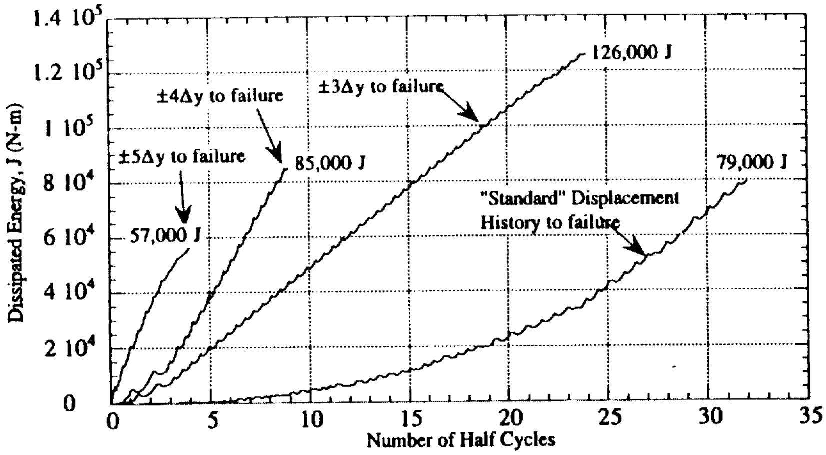

Figure 8: Cumulative energy to failure for Specimens 2, 4, 5 and 6

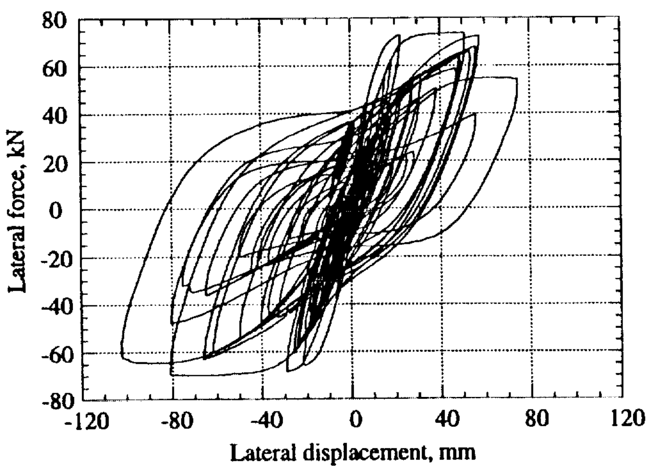

Figure 9: Lateral load-displacement plot for Specimen 12

Another significant observation from the constant amplitude tests was that cumulative dissipated energy (the area contained within the hysteresis loops) is not good predictor of column failure. It was found that the cumulative dissipated energy at failure depended on the amplitude of the sawtooth wave. Figure 8 shows the accumulation of energy to failure for Specimens 2, 4, 5 and 6 ("standard" displacement pattern, $\pm 3\Delta y$, $\pm 4\Delta y$, and $\pm 5\Delta y$, respectively). Results for Specimen 3 are not plotted because the specimen did not deteriorate significantly under 150 cycles at $\pm 2\Delta y$, so a failure state was not achieved. In Figure 8 it can be seen that dissipated energy at failure is strongly dependent on the displacement history. Therefore, because of the highly variable nature of earthquake-induced displacement histories, it is apparent that cumulative energy alone is not an acceptable measure of column damage.

3.2 *Earthquake Loading Tests*

A typical lateral load-displacement plot for one of the earthquake loading tests is shown in Figure 9. It was observed that failure of the columns could be classified into two general types: failure due to low cycle fatigue of the longitudinal reinforcement, and failure due to rupture of confining reinforcement. While these two classes of failure have been observed, it is interesting to recall that the only variable in these tests was the displacement history. Thus, displacement history, rather than the column configuration, determined the failure mode.

4. CONCLUSIONS

This paper examined the influence of testing protocol in determining the inelastic structural response factor such as ductility in establishing performance criteria for structural members. Quantitative values which represent structural response factors are dependent highly upon the cyclic loading history to which structural members are subjected. The followings are specific findings of this study.

1. There appears to be a threshold ductility level above which degradation of stiffness and strength is rapid. For the columns tested in this study, this threshold ductility level was about $4\Delta y$. Below this threshold degradation was more gradual, and for ductility demands of $2\Delta y$ or less degradation was minimal. If the degree of structural damage is to be expressed in terms of "ductility," it is important to establish a ductility level for a structural system above which a structure would experience rapid degradation.

2. The constant amplitude tests in this study confirm that cumulative dissipated energy at failure is strongly path dependent. Therefore, cumulative energy by itself is not a reliable measure of structural damage, and should not be used to index damage level.

3. The random loading tests indicate that failure mode is path dependent. Nominally identical columns failed either due to low-cycle fatigue of longitudinal reinforcement, or due to rupture of confining reinforcement. The only variable between tests was the load path. Failure mode is dependent upon the loading history to which a structure is subjected.

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