EVALUATION OF DUCTILITY OF STRUCTURES AND STRUCTURAL ASSEMBLAGES FROM LABORATORY TESTING

R. Park*

ABSTRACT

Definitions for the required and available ductility used in seismic design are discussed. Methods for estimating the yield deformation and the maximum available deformation are described and suggestions are made for appropriate definitions. Examples are given of different imposed histories of inelastic displacement which have been used in the experimental testing of structures and structural assemblages in which cycles of quasi-static loading are applied. A quasi-static procedure for establishing the available ductility factor of a subassemblage by laboratory testing is recommended.

1. INTRODUCTION

The term "ductility" in seismic design is used to mean the ability of a structure to undergo large amplitude cyclic deformations in the inelastic range without a substantial reduction in strength. Ductile structures are generally able to dissipate significant amounts of energy during those cyclic deformations. The required ductility of a structure responding to a severe earthquake can be estimated analytically by nonlinear time-history dynamic analysis or more approximately by the consideration of static mechanisms of inelastic deformations. Alternatively the required ductility of a structure responding to a severe earthquake can be estimated experimentally by shaking table tests or pseudodynamic tests. The ductility of a structure should be such as to ensure that the available ductility is at least equal to the required ductility.

This paper considers definitions which enable the determination of the required and available ductility, and methods for evaluating the available ductility of structures and structural assemblages from laboratory tests.

2. THE REQUIRED DUCTILITY FACTOR

In the nonlinear time-history dynamic analysis of structures responding to a severe earthquake in the inelastic range it is usual to express the maximum deformations in terms of ductility factors, where the ductility factor is defined as the maximum deformation divided by the corresponding deformation when yielding occurs. The use of ductility factors permits the maximum deformations to be expressed in nondimensional terms as indices of inelastic deformation for seismic design and analysis. Ductility factors have been commonly expressed in terms of the various response parameters related to deformations, namely the displacements, rotations and curvatures.

The displacement ductility factor, $\mu = \Delta_{max}/\Delta_y$, where Δ_{max} is the maximum displacement and Δ_y is the displacement at yield, is the value normally determined in inelastic time-history dynamic analyses. The displacement ductility factor μ is shown defined for ideal elasto-plastic behaviour in Fig.1.

The displacement ductility factor required of typical code-designed structures may vary between 1 for elastically responding structures to 6 for ductile structures, depending on the level of seismic design force used to determine the required strength of the structure.

Analytical approaches also commonly determine the rotation ductility factor required of members θ_{max}/θ_y, where θ_{max} is the maximum rotation at the plastic hinge and θ_y is the rotation in the plastic hinge region at yield.

The information most needed by structural designers is the required curvature behaviour of the critical sections of members in plastic hinge regions, expressed by the curvature ductility factor ϕ_{max}/ϕ_y, where ϕ_{max} is the maximum curvature at the section and ϕ_y is the curvature there at yield.

For structures in which ductility is controlled by flexural plastic hinging of members the available displacement

* Professor and Head of Department of Civil Engineering, University of Canterbury, New Zealand

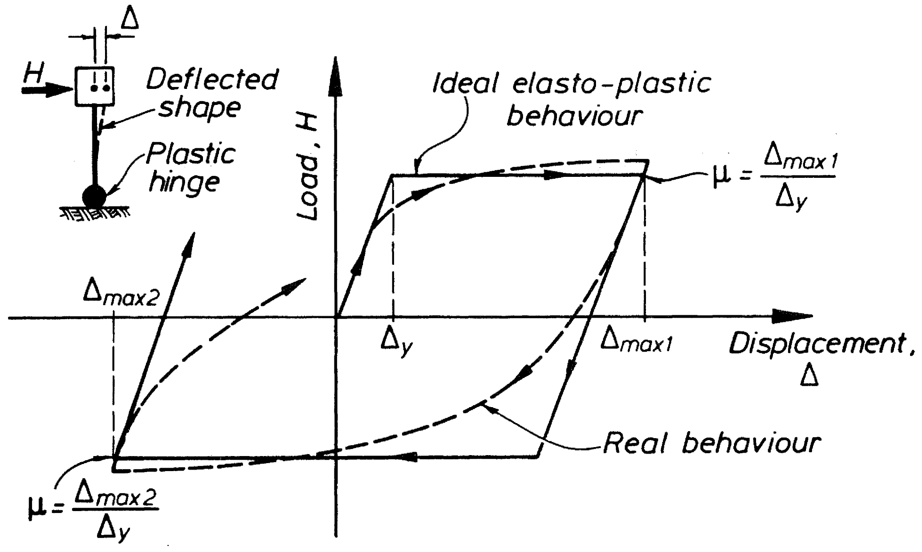


Fig.1 DISPLACEMENT DUCTILITY FACTOR

ductility factor will be limited by the available (ultimate) curvature ductility factor. The relationship between the displacement ductility factor of a reinforced concrete structure and the curvature ductility factors at the plastic hinges can be determined considering the geometry of the deformation of the structure, providing that the equivalent plastic hinge length, over which the ultimate curvature can be considered constant [1,2], is known. In recent reinforced concrete column tests [2] the equivalent plastic hinge length, taking into account the spread of plasticity due to bond deterioration and diagonal tension cracking, was found experimentally to be on average close to $\ell_p = 0.5h$, where h is the column depth. The plastic hinge rotation is given by $\theta_p = (\phi_u - \phi_y)\ell_p$.

It is evident that there can be significant numerical differences between the magnitudes of the required displacement, rotation and curvature ductility factors. This is because once yielding has commenced in a structure the deformations concentrate in the yielding regions. For example, for reinforced concrete moment resisting frames the required ϕ_{max}/ϕ_y at the plastic hinges may be several times the required Δ_{max}/Δ_y for the structure [1].

3. EFFECT OF HYSTERESIS LOOP SHAPE ON RESPONSE

Fig.1 illustrates that the load-deformation behaviour of real members can vary significantly from ideal elasto-perfectly plastic behaviour. Fig.2 shows a range of typical measured experimental lateral load-displacement loops for subassemblages of structural concrete, masonry, steel and timber. A number of shapes of hysteresis loops have been used to model the cyclic load-deformation behaviour of structures of different materials to be utilized in inelastic time-history dynamic analyses, such as bilinear with variable post-yield stiffness, and more complex stiffness degrading idealizations which closely follow the actual loop shapes. Several investigators have studied the influence of the shape of the hysteresis loops on the response of structures to severe earthquakes.

Of particular interest is the effect on the response of significant stiffness degradation when the structure is cycled in the inelastic range. On average, the difference between the ductility demands for elasto-perfectly plastic single-degree-of-freedom systems and stiffness degrading systems with the same initial strength, initial stiffness and viscous damping, when responding to severe earthquakes found by Mahin and Bertero [8] and Moss, et al [9] were small, except perhaps for short period structures where the ductility demand of the degrading stiffness system may be larger. Degrading stiffness systems were found to dissipate hysteretically about the same amount of energy as elasto-perfectly plastic systems, even though they do not reach their full strength as often [8]. This is because energy is dissipated hysteretically by the elasto-perfectly plastic system only when the full strength is reached, but for the stiffness degrading system (for example, shown as real behaviour in Fig.1) energy is dissipated due to non-linear behaviour in almost all cycles after first yield.

However Mahin and Bertero [8] have found that bilinear hysteretic loops with even a small negative post-yield slope (-5%) can substantially increase the ductility demand, particularly for short period structures and long duration earthquakes. It should be noted though that the bilinear model is not typical of real behaviour of structural members. Stiffness degrading models are more typical and the reduction of strength seen in hysteresis loops generally occurs as an overall reduction in strength (as in Fig.2) or only at the end

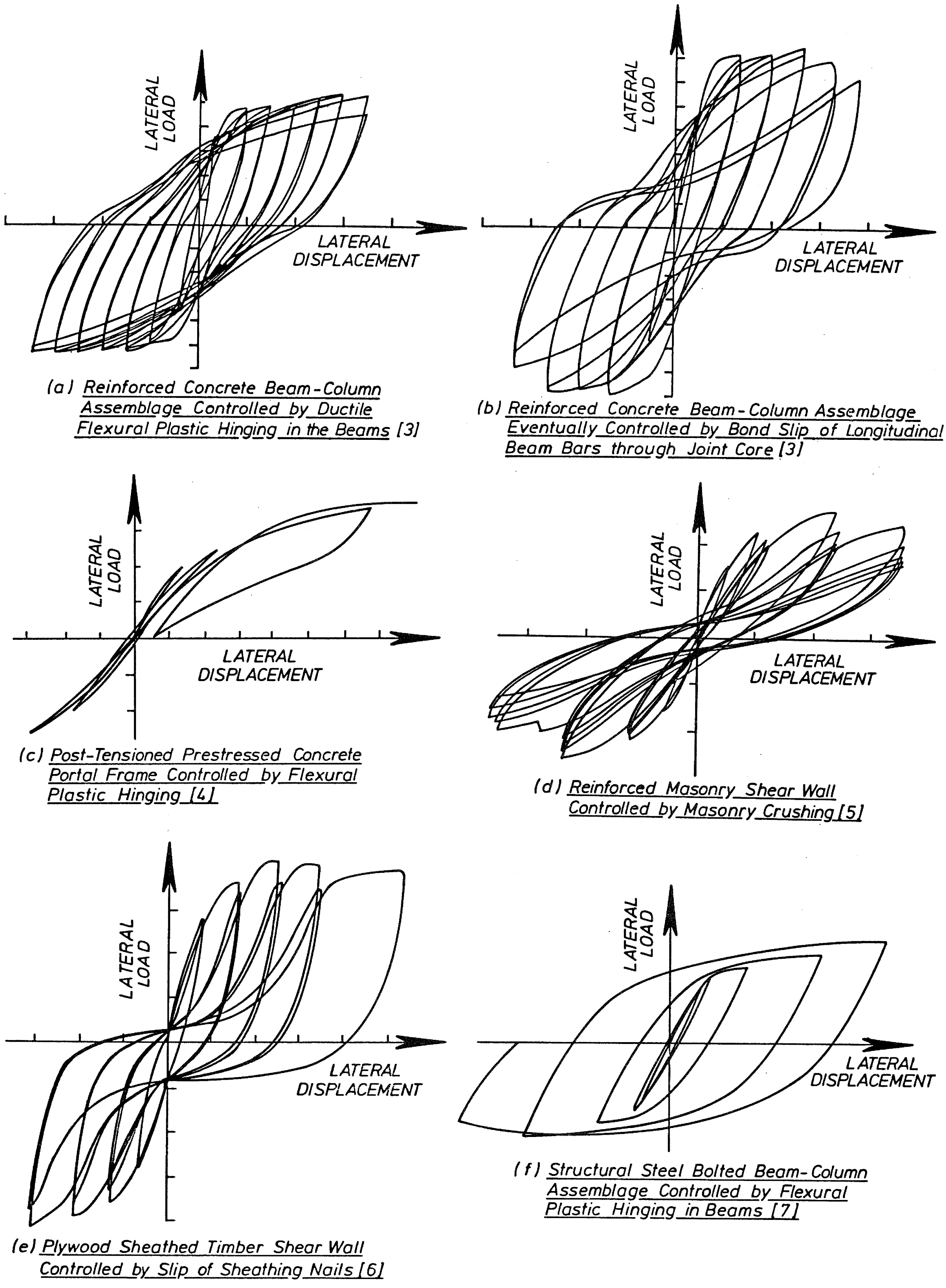


Fig.2 TYPICAL MEASURED LATERAL-LOAD DISPLACEMENT HYSTERESIS LOOPS FOR SUBASSEMBLAGES OF STRUCTURAL CONCRETE, MASONRY, STEEL AND TIMBER

of the post-yield load-deformation branch. Moss, et al [9] have found using elasto-perfectly plastic hysteresis loops that strength degradation to 80% of the initial strength during severe seismic excitation in the inelastic range did not significantly influence the displacement response.

Prestressed concrete members have significantly narrower moment-curvature hysteresis loops (see Fig.2c), and hence very much lower hysteretic energy dissipation, than reinforced concrete or structural steel members. The maximum displacements reached by code-designed prestressed concrete single-degree-of-freedom systems has been found to be on average approximately 30% greater than reinforced concrete systems of similar initial strength, initial stiffness and viscous damping, when responding to severe earthquakes [10].

For reinforced concrete structures significant inelastic deformations due to shear or bond mechanisms lead to severe degradation of strength and stiffness and to pinched hysteresis loops with reduced energy dissipation. Fig.2a and 2b shows typical measured experimental load-displacement hysteretic behaviour of two reinforced concrete beam-column assemblies [3], one controlled by ductile flexural plastic hinging in the beams (Fig.2a) and the other controlled eventually by slip of longitudinal beam bars through the joint core due to bond deterioration (Fig.2b). Kitayama, et al [11] have investigated the inelastic dynamic response to severe earthquake motions of 4, 7 and 16 storey moment resisting frames with the plastic hinge behaviour in the beams modelled by stiffness degrading hysteresis loops with and without pinching behaviour caused by bond deterioration. The effect of significant pinching of the hysteresis loops on the response was found to be relatively small and it was concluded that some bond deterioration of beam bars within a beam-column joint may be tolerable.

Similarly, Dean et al [6] found that the displacement demand for degrading stiffness structures with relatively low energy dissipating capacity, such as plywood sheathed timber shear walls (see Fig.2e), to severe earthquake motions is not significantly greater than that for structures responding elasto-plastically.

It is evident that in the past there has been excessive emphasis on the desirability of achieving in design structures which, when subjected to cyclic deformations in the inelastic range due to severe earthquake loading, display "fat" load-deformation hystereis loops. It is now realized that some variation in hystersis loop shape will not have a major influence on the inelastic dynamic response of structures when subjected to severe earthquake excitation. That is, hysteresis loops showing some pinching or stiffness degradation will not lead to significantly larger inelastic displacements, providing that the structure has some damping of viscous type and is capable of some further damping by hysteretic energy dissipation.

It is concluded that the important property required in seismic design is "adequate ductility", which means the ability of the structure to undergo large amplitude cyclic deformations to the required maximum displacement in the inelastic range without a substantial reduction in strength.

4. DEFINITIONS OF AVAILABLE DUCTILITY

The ductility required of a structure during response to a severe earthquake needs to be matched by the available ductility of the structure. Definitions which can be used to estimate the available ductility factor are considered below.

4.1 Definition of the Yield Deformation

When calculating ductility factors the definition of the yield deformation (displacement, rotation or curvature) often causes difficulty since the load-deformation relation may not have a well defined yield point. This may occur, for example, due to nonlinear behaviour of the materials or due to yielding in different parts of a structure commencing at different load levels.

Various alternative definitions which have been used by investigators to estimate the yield displacement are illustrated in Fig.3. These are :

Fig.3a : The displacement when yielding first occurs in the system.

Fig.3b : The yield displacement of the equivalent elasto-plastic system with the same elastic stiffness and ultimate load as the real system.

Fig.3c : The yield displacement of the equivalent elasto-plastic system with the same energy absorption as the real system [8].

Fig.3d : The yield displacement of the equivalent elasto-plastic system with reduced stiffness found as the secant stiffness at either first yield or at 0.75 of the ultimate lateral load H_u, whichever is less [2]. The non-linear elastic behaviour before first yield or 0.75 H_u is due to cracking in the case of reinforced concrete.

The definition illustrated in Fig.3d is considered to be the most realistic since it applies generally to structures of concrete, masonry, steel and timber.

4.2 Definition of the Maximum Available (Ultimate) Deformation

The maximum available (ultimate) deformation has also been estimated using various assumptions by investigators in the past. Some possible definitions for the maximum available displacement are shown in Fig.4. These are :

Fig.4a : The displacement corresponding to a particular limiting value for the compressive strain. (For

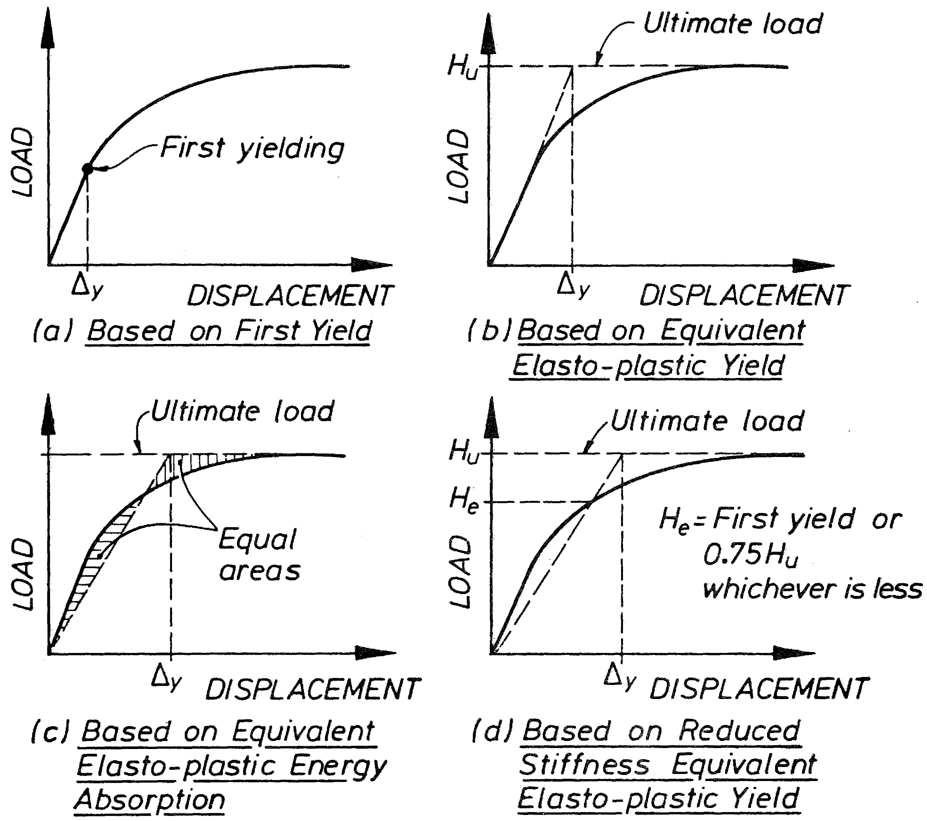


Fig.3 ALTERNATIVE DEFINITIONS FOR YIELD DISPLACEMENT

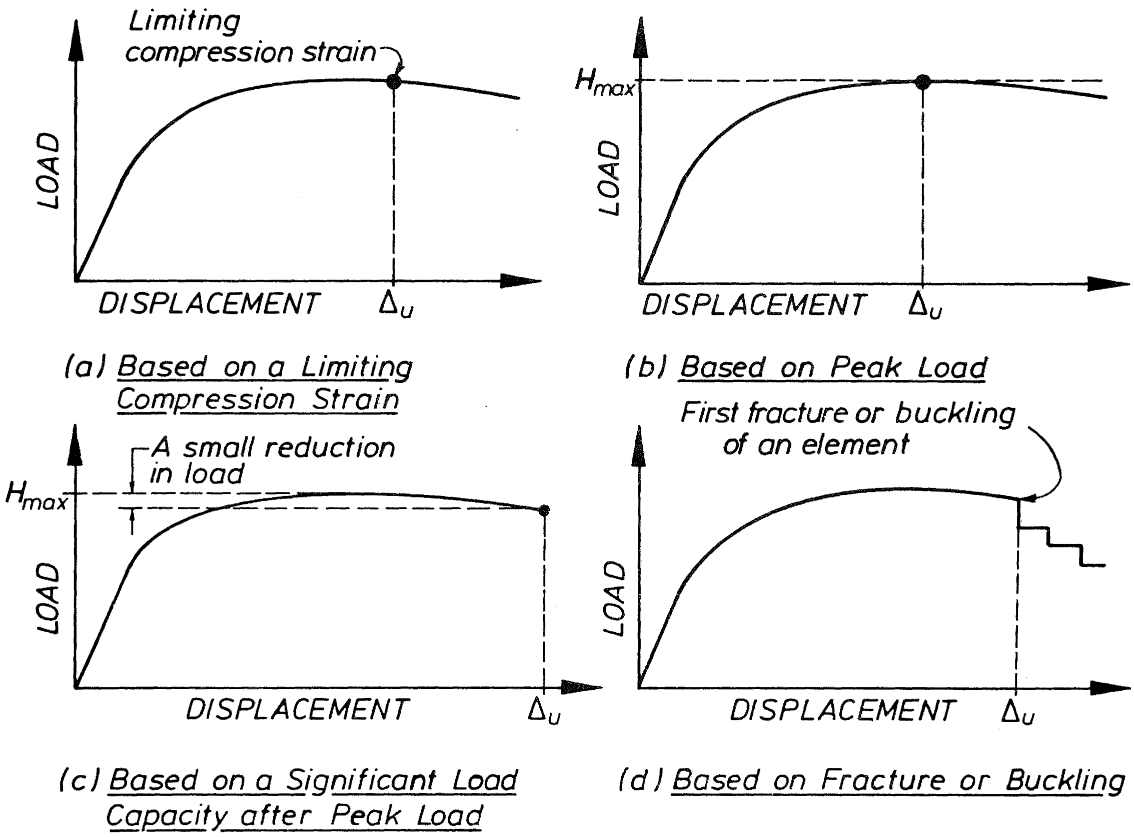


Fig.4 ALTERNATIVE DEFINITIONS FOR MAXIMUM AVAILABLE (ULTIMATE) DISPLACEMENT

example, the attainment of a specified "ultimate" compressive strain in the case of concrete).

Fig.4b : The displacement corresponding to the peak of the load-carrying capacity.

Fig.4c : The post-peak displacement when the load carrying capacity has undergone a small reduction (For example, a 20% reduction in load).

Fig.4d : The displacement when the material fractures or elements buckle. (For example, in the case of reinforced concrete when the transverse or longitudinal reinforcing steel fractures or the longitudinal compression reinforcement buckles.

When considering the most appropriate definition it should be recognized that most structures have some capacity for deformation beyond the peak of the load-displacement relation without a significant reduction in strength. It would be reasonable to recognize at least part of this post-peak deformation capacity. Also, it is evident that for reinforced concrete the maximum available deformation does not necessarily correspond to a specified extreme fibre concrete compresssive strain [1].

Hence the most realistic definition for the maximum available displacement is considered to be that given by the criteria shown in Figs. 4c and 4d, whichever occurs first.

The definition for the maximum available deformation could also include a cyclic loading parameter, such as the maximum deformation when after a specified number of cycles of loading to that deformation the load carrying capacity has reduced by a small specified amount or the material has fractured or elements have buckled.

4.3 Definition of Available Ductility Factor

The available displacement ductility factor, rotation ductility factor and curvature ductility factor can be written as Δ_u/Δ_y, θ_u/θ_y and ϕ_u/ϕ_y, respectively, where the maximum available (ultimate) and yield quantitites are defined as in Figs.3 and 4.

4.4 Cumulative Ductility Factor

The cumulative ductility factor undergone by a structure during cycles of loading is also of interest when assessing the effects of several cycles of loading. For example, a structure subjected to 4 cycles of loading to displacement ductility factors of 4 in each direction would undergo a cumulative displacement ductility factor of $\Sigma\mu = 32$. Care should be taken when assessing the effects on the structure of cumulative ductility factors. For example, 16 cycles of loading to $\mu = 1$ in each direction can result in significantly less damage to the structure than 2 cycles to $\mu = 8$ in each direction, although both loading histories give a cumulative displacement ductility factor of $\Sigma\mu = 32$.

5. EXPERIMENTAL METHODS FOR DUCTILITY EVALUATION

The experimental testing of structures and structural assemblages in laboratories, to assess performance and available ductility during severe earthquakes, requires decisions concerning the appropriate displacement history to be imposed to simulate seismic loading. The three types of seismic load testing used in experimental studies are briefly described below.

5.1 Shake Table Testing

Shake table testing, with the table following the motions of a recorded earthquake at dynamic strain rates, is a realistic experimental method for assessing the performance and the required and available ductility of structural systems. A major limiting factor is the mass, size and strength of structure that can be tested since these will depend on the capacity of the shake table. Often only scale models can be tested and scaling of the earthquake record may also be necessary. Also the equipment and instrumentation required for realistic shake table testing may be unavailable in many laboratories.

5.2 Pseudodynamic Testing

Pseudodynamic testing is an alternative which retains the realism of shake table testing but has the convenience of conventional quasi-static loading tests [12]. In pseudodynamic testing experimental measurements are made of the restoring forces of the structure at each step during the testing, and this direct experimental feedback is used to calculate by inelastic dynamic computer analysis the displacements to be imposed on the structure in the next step by hydraulic actuators to closely resemble those that would occur if the structure was subjected to the ground shaking of a particular earthquake.

5.3 Quasi-Static Cyclic Load Testing

Most experimental testing of structures and structural assemblages has used quasi-static cyclic loading, applied by hydraulic actuators, which has not attempted to follow the strain rate or the specific displacement history imposed by a particular earthquake. Instead the structure is subjected to predetermined numbers of displacement controlled quasi-static loading cycles to predetermined displacement ductility factors. The slow strain rate means that the test may take several days to conduct.

Time-history dynamic analyses of code-designed structures responding inelastically to major earthquakes can be used to obtain a guide as to the quasi-static loading history to be applied. For example, Mahin and Bertero [13] have found that the number of yield excursions tends to increase with decreasing period of vibration, except in the very short period range. Yielding was found to occur about the same number of times in each direction, but the maximum displacement was generally larger in one direction than in the other. For stiffness degrading single-degree-of-freedom-systems, designed for a seismic force level corresponding to a displacement ductility factor of 4, responding to severe earthquake ground motions recorded on firm ground at moderate epicentral distances, such as the 1940 El Centro earthquake, the number of yield reversals in each direction did not generally exceed 4. For the unrealistic elasto-perfectly plastic systems the number of yield excursions in each direction varied from about 15 for a period of 0.2 seconds to about 3 for a period of about 2 seconds. It is to be noted that the destructive earthquake which occurred in Mexico City in September 1985 had most of its energy in relatively long period ground motions and the duration of the strong earthquake shaking was exceptionally long. Hence the number of yield excursions for that Mexico City earthquake would be expected to be several times that of the 1940 El Centro earthquake.

Quasi-static load testing gives conservative estimates of the real strength of the structure or structural assemblage, since real earthquake loads are dynamic and an increase in the strain rate results in an increase in the strength of the materials. However significant differences between the shapes of the hysteresis loops obtained from quasi-static and dynamic loading tests may not be observed. For example Iwasaki, et al [14] concluded that the effect of loading velocity on the energy dissipation of reinforced concrete columns was not significant for displacements of 4 times that at first yield of the longitudinal reinforcement, but at higher displacements the energy dissipation capability was appreciably larger when the loading velocity was 100 cm/sec than when the loading velocity was 10 cm/sec.

In quasi-static load testing the displacement history does not follow in detail the complex response of a structure to an actual earthquake. Instead a more simple displacement history is applied to enable an assessment to be made as to whether the structure is tough enough to be likely to perform satisfactorily during a severe earthquake. Unfortunately, investigators in the past have used a range of displacement histories, and various definitions of yield and ultimate deformation, which have made the comparison of results of different investigations difficult. As a result, values for ductility factor obtained from experimental tests have sometimes been misused in judging the likely performance of structures during severe earthquakes.

Agreement is needed for appropriate definitions of the main parameters describing inelastic behaviour for quasi-static load testing, so that performance obtained from analytical and experimental investigations can be properly assessed and compared in terms of their application to the design of structures for earthquake resistance.

5.4 Examples of Quasi-Static Cyclic Loading in Terms of Displacement Ductility

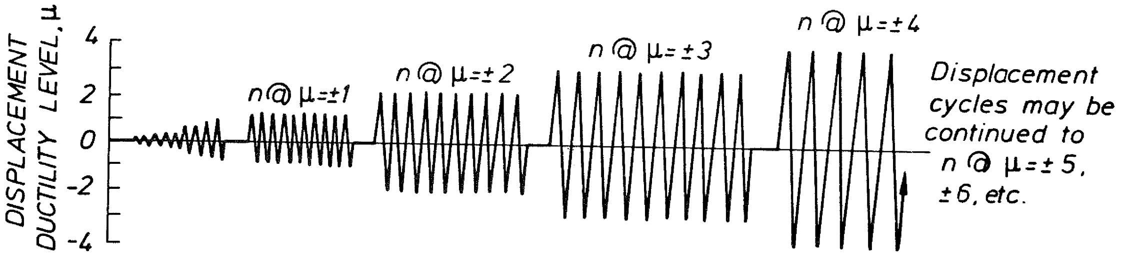
As an example of a quasi-static loading history, the Commentary of the current (1984) SANZ Code for general design and loadings for buildings [15] recommends, as an approximate criterion for the adequate ductility of moment resisting frames, that the structure should be able to undergo 4 loading cycles to a displacement ductility factor of 4 in each direction without the horizontal load carrying capacity reducing by more than 20%.

A quasi-static loading pattern which has been used for tests at the Construction Technology Laboratories, Skokie, USA [16] and at the Public Works Research Institute, Ministry of Construction, Japan [14], and at several other organizations, is shown in Fig.5a. The displacement Δ_y has been taken as the displacement corresponding first yield of the outer longitudinal reinforcing bars. The ductility level is increased step-wise and the number of symmetrical loading cycles at each ductility level has been n = 2 to 10, typically n = 2 in the United States [16], and n = 10 in Japan [14].

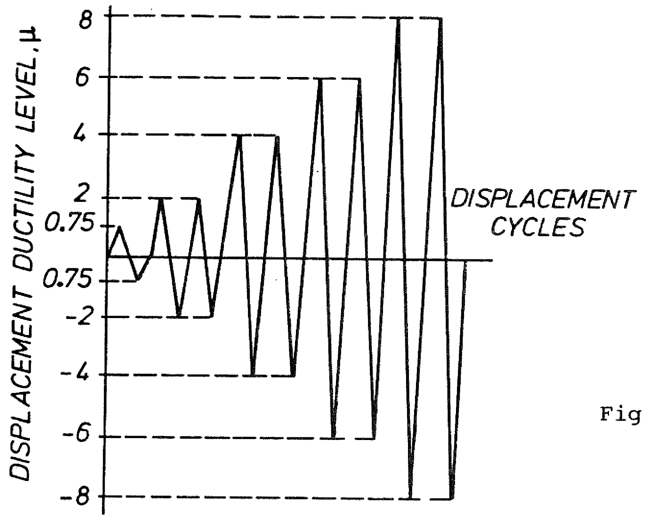
A quasi-static loading pattern which has been used for many years at the University of Canterbury [2] is shown in Fig.5b. The yield displacement is found using the mean measured secant stiffness at either 0.75 of the theoretical ultimate load or at first yielding of steel, whichever is least, as illustrated in Fig.3d. Again the ductility level is increased step-wise and normally two symmetrical loading cycles have been applied at each level. Sometimes the ductility levels have been increased in steps of 2 cycles to $\mu = \pm 1, \pm 2, \pm 3$ etc., if limited ductility is expected. In New Zealand the commonly used strength criterion is that the reduction in strength should not exceed 20% of the initial strength.

A more detailed quasi-static loading history used for seismic load tests involving bi-directional earthquake loading is that agreed to by the principal investigators of the United States-New Zealand-Japan-China collaborative research project on the seismic design of reinforced concrete beam-column joints [17]. Again the yield displacement is determined from the secant stiffness measured at 0.75 of the theoretical ultimate load. The displacement controlled loading history imposed is illustrated in Fig.6 for the first 12 cycles. Obtaining that international agreement was a major step forward and will permit proper comparison of the performance of the structures tested in the four countries.

The above quasi-static loading histories used in New Zealand and in the US-NZ-Japan-China collaborative research project are suitable for earthquakes of typical



(a) Tests at the Construction Technology Laboratories, Skokie, USA and at the Public Works Research Institute of the Ministry of Construction, Tsukuba, Japan.



(b) Tests at the University of Canterbury

Fig.5 EXAMPLES OF DISPLACEMENT HISTORIES USED FOR QUASI-STATIC CYCLIC LOADING TESTS OF COLUMNS AND STRUCTURAL ASSEMBLAGES

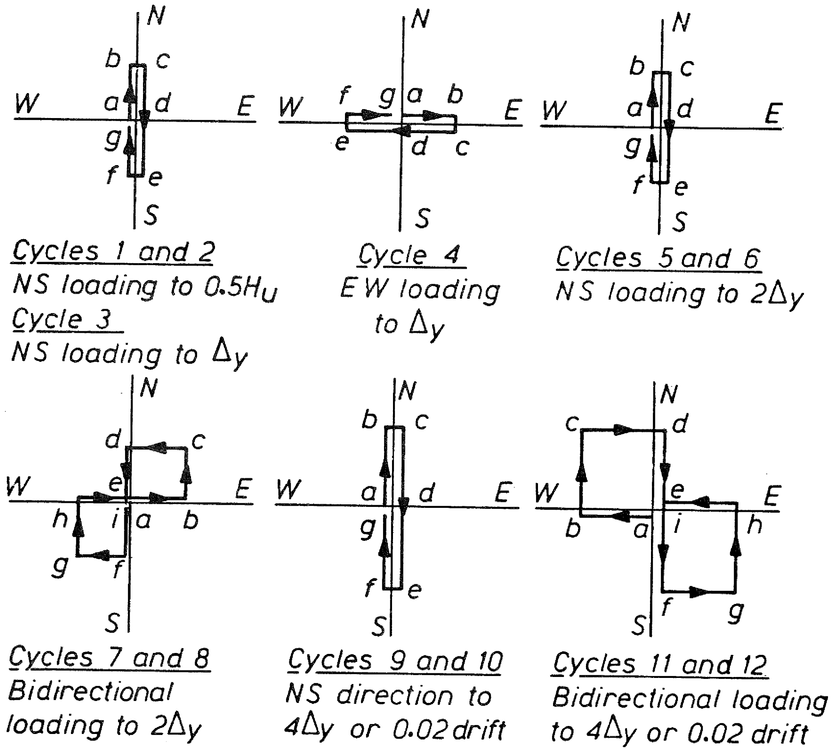


Fig.6 BIDIRECTIONAL DISPLACEMENT HISTORY USED FOR QUASI-STATIC CYCLIC LOADING TESTS OF THE US-NEW ZEALAND-JAPAN-CHINA COLLABORATIVE RESEARCH PROJECT ON REINFORCED CONCRETE BEAM-COLUMN JOINTS [17]

duration. For a long duration earthquake, such as the September 1985 Mexico City event, more loading cycles at each ductility level would be necessary.

In quasi-static loading tests, care should be taken to ensure that the test structure or structural subassemblage is adequately stiff to satisfy the code limitations for interstorey displacements. If the test structure or assemblage is overly flexible the level of interstorey displacement required to achieve a given displacement ductility level may be unrealistically large.

5.5 Quasi-Static Cyclic Loading in Terms of Interstorey Drift

It has been suggested by some investigators that the imposed deformation history should be based on the level of interstorey drift rather than on the level of displacement ductility factor. The interstorey drift is obtained by dividing the interstorey horizontal displacement by the storey height.

Interstorey drift, rather than displacement ductility factor level, is commonly used in quasi-static loading tests in the United States and Japan. For example, in the United States, Zhu and Jirsa [18] have suggested that if the test structure or structural subassemblage can withstand imposed displacement cycles with interstorey drifts of up to ±3% without substantial loss in strength, the structure is satisfactory. In Japan interstorey drifts of up to ±2% are commonly imposed in tests.

The concept of using interstorey drift as a test criterion has considerable merit since it avoids the difficulty of the definition of the yield displacement. However care should be taken in the use of interstorey drift as a test criterion since the level of imposed interstorey drift should depend on the stiffness of the structure and the level of displacement ductility factor to be imposed, as found from dynamic analysis. For example, the code specified limitation for interstorey drift, for a structure behaving elastically when subjected to the code designed lateral force, may be 0.35%. If a structural assemblage has this limiting elastic stiffness and a displacement ductility factor of $\mu = 6$ is imposed, the resulting interstorey drift will be 0.35 x 6 = 2.1%. However, if the structural assemblage is stiffer than the code minimum, a 2.1% imposed interstorey drift will result in an imposed displacement ductility factor of much higher than 6. Hence the imposed interstorey drift should depend on the stiffness of the structure and on the required ductility.

Also, measured interstorey drifts do not give an indication of the available ductility factor of the structural assemblage. Ideally, hysteretic responses measured in quasi-static load tests when plotted should have marked on them both the displacement ductility factor and the interstorey drift.

6. A RECOMMENDED QUASI-STATIC LOADING TEST PROCEDURE TO ESTABLISH A DUCTILITY FACTOR FOR A SUBASSEMBLAGE

6.1 The Need for a Test Procedure

One of the important variables used in establishing the level of design seismic loading is the available ductility of the structure. The recent draft SANZ code for general design and design loadings for buildings [19] uses structure (displacement) ductility factors for assessing the appropriate response spectrum to use with the equivalent static or modal response methods of analysis. Structure ductility factors are recommended in the draft code [19] for conventional materials and structural forms.

In the development of new design or construction procedures, or materials, or structural forms, it will be necessary to determine appropriate structure (displacement) ductility factors.

The appropriate structure (displacement) ductility factor for a particular structural subassemblage can be found by a test which establishes that the subassemblage can sustain not less than 80% of the measured maximum strength when subjected to four cycles of loading to a selected ductility level. Such a test has the disadvantage that the subassemblage either passes or fails the criterion at that particular ductility level. To establish the actual available (displacement) ductility factor by this method a large number of tests are required.

6.2 A Recommended Test Procedure

The procedure given below is presented as an alternative. It has the advantage that each test yields a structure (displacement) ductility factor value for the subassemblage. It is based on test criteria (Fig.3d and 5b) which have commonly been used at the University of Canterbury.

The test is carried out in two parts:

Part 1 Load controlled test cycles.

The lateral load levels required to cause first yield in the subassemblage, and to cause the the ideal strength of the subassemblage to be reached, in each direction, are calculated. These values are based on the measured material properties, a strength reduction factor of 1.0 and the analytical procedure used in design.

Lateral loading is applied in one direction to the subassemblage and increased until reaching the calculated or measured value required to initiate first yielding or three quarters of the calculated ideal strength, whichever is less. The lateral loading is then applied in the opposite direction to first yield or three quarters of ideal strength, whichever is less. Straight lines are fitted to the load-displacement measurements in each direction, passing through the origin and the displacement points at first yield or three quarters of the ideal strength, whichever is less. These lines are extended until they intersect the load corresponding to the ideal strength values. The mean of the displacements corresponding to these intersection points in the two directions of loading is taken as the yield displacement Δ_y (see Fig.7). This in fact is the yield displacement of an equivalent elasto-plastic system with reduced elastic stiffness to take into account the effect of cracking and other non-linear elastic effects.

Part 2 Displacement controlled test cycles

In the second part of the test the imposed structure (displacement) ductility factor μ is increased step-wise in cycles, where $\mu = \Delta_{max}/\Delta_y$ and Δ_{max} is the maximum displacement imposed in the loading excursion. First, two cycles to $\mu = \pm 2$ are applied, followed by two cycles to $\mu = \pm 4$ and two cycles to $\mu = \pm 6$. Beyond this level further cycles to $\mu = \pm 6$ or ± 8 may be applied. The displacement history is shown in Fig.8. In the figure the cumulative structure ductility factor $\Sigma\mu$ undergone during the cycles of displacements is shown at the peak of each loading run. For example, a subassemblage which has been subjected to two cycles to $\mu = \pm 2$ followed by one cycle to $\mu = \pm 4$ would have undergone a cumulative structure ductility factor of $\Sigma\mu = 16$.

The measured load-displacement record of the subassemblage is then assessed to determine the available structure (displacement) ductility factor μ_a. This is carried out by determining the cumulative structure ductility factor $\Sigma\mu$ at which the lateral load sustained has reduced to 80% of the maximum applied lateral load strength measured during the test. The available structural ductility factor is then taken as $\mu_a = \Sigma\mu/8$. This procedure assumes that four load cycles to μ_a (that is, eight load excursions) will result in the same reduction in applied load as the history of displacement cycles shown in Fig.8 applied to the appropriate $\Sigma\mu$, where $\Sigma\mu = 8\mu_a$.

For example, if in the test the applied lateral load in load excursion "m" (see Fig.8) is for the first time less than 80% of the maximum measured lateral load during the whole load excursion, the available cumulative ductility is $\Sigma\mu = 36$. Then the available structure ductility factor is $\mu_a = 36/8 = 4.5$.

As a further example, if in the test the applied lateral load reduces for the first time to 80% of the maximum measured lateral load strength at $\mu = +3$ in load excursion "m" (see Fig.8), the available cumulative ductility factor is $\Sigma\mu = 36 + 3 = 39$. Then the available structure ductility factor is $\mu_a = 39/8 = 4.9$.

Note that in this test the main concern is the level of ductility reached before significant reduction in applied load occurs, rather than the energy dissipated by the assemblage as given by the area within the hystersis loops. This is because, as discussed in Section 3, the energy dissipated by the structure is a less important factor in seismic design than the displacement capacity.

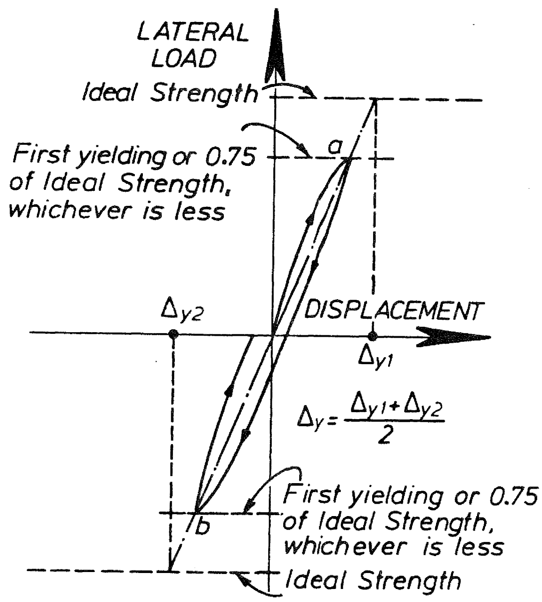


Fig.7 DEFINITION OF YIELD DISPLACEMENT

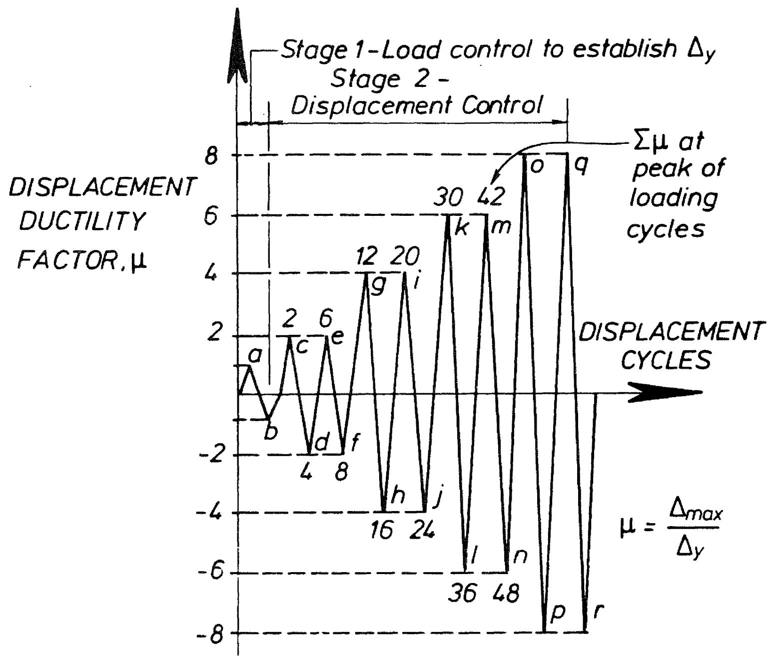


Fig.8 IMPOSED DISPLACEMENT HISTORY USED TO ESTABLISH THE AVAILABLE STRUCTURE (DISPLACEMENT) DUCTILITY FACTOR

7. CONCLUSIONS

1. Ductility factors of structures, expressed as the maximum deformation divided by the corresponding deformation at yield, are useful nondimensional indices of inelastic deformations. Ductility factors can be defined in terms of the required ductility during severe earthquakes and the available ductility, and can be expressed in terms of displacements, rotations and curvatures.

2. Values of ductility factor have sometimes been misused in the past due to the various current definitions for the yield deformation and the maximum available (ultimate) deformation for structural subassemblages when the shapes of the load-deformation hysteresis loops are not elasto-perfectly plastic. Agreement is needed as to the definitions of "yield" and "maximum available (ultimate)". It is suggested that the yield deformation should be estimated from an equivalent elasto-perfectly plastic system with elastic stiffness which includes the effects of cracking, and other nonlinear elastic effects, and with the same ultimate load as the real system. The maximum available (ultimate) deformation should be estimated as that post-peak deformation when the load-carrying capacity has reduced by a small specified amount, or when fracture of material or buckling of elements occurs, whichever is least.

3. In the past experimental testing of structures and structural assemblages in which cycles of quasi-static loading are applied, has involved the use of many different inelastic displacement histories. The quasi-static loading history agreed to by the principal investigators of the US-NZ-Japan-China collaborative research project on the seismic design of reinforced concrete beam-column joints, for testing involving bidirection earthquake actions. represents a substantial step forward in agreement.

4. Experimental testing using imposed displacements, based on the percentage of the storey height (that is, the interstorey drift) rather than displacement ductility factor, has considerable merit since it avoids the difficulty of defining the yield deformation. However the interstorey drift does not give an indication of the available ductility of the system since the available ductility will also depend on the elastic stiffness of the system.

5. A quasi-static loading procedure is recommended for laboratory tests. The test procedure can be used to determine an appropriate structure (displacement) ductility factor for subassemblages when new design or construction procedures, or materials, or structural forms, are being investigated. In the test procedure the imposed structure (displacement) ductility factor is increased step-wise in cycles. The measured cummulative ductility factor, up to the stage when the lateral load sustained has decreased to 80% of the maximum measured lateral load strength, is used to calculate the available structure (displacement) ductility factor.

ACKNOWLEDGEMENTS

Informative discussions with colleagues, particularly Professor T. Paulay of the University of Canterbury, Professor M.J.N. Priestley of the University of California at San Diego and Dr R.C. Fenwick of the University of Auckland, are gratefully acknowledged.

REFERENCES

1. R. Park, and T. Paulay, "Reinforced Concrete Structures", John Wiley, New York, 1975, p.769.

2. M.J.N. Priestley, and R. Park, "Strength and Ductility of Concrete Bridge Columns Under Seismic Loading", Structural Journal, American Concrete Institute, Vol.84, No.1, Jan-Feb.1987, pp. 61-76.

3. R. Park and Dai Ruitong, "A Comparison of the Behaviour of Reinforced Concrete Beam-Column Joints Designed for Ductility and Limited Ductility", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol.21, No.4, December 1988, pp.255-278.

4. K. Nakano, "Experiments on the Behaviour Under Lateral Force of Prestressed Concrete Portal Frames", Transactions of the Japan Architectural Institute, No.102, September 1964. (In Japanese).

5. M.J.N. Priestley and D. McG. Elder, "Cyclic Loading Tests on Slender Concrete Masonry Shear Walls", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol.15, No.1, March 1972, pp.3-21.

6. J.A. Dean, W.G. Stewart and A.J. Carr, "The Seismic Behaviour of Plywood Sheathed Shear Walls", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol.19, No.1, March 1986, pp.48-63.

7. N.D. Johnson and W.R. Walpole, "Behaviour of Steel Beam-Column Connections with Bolted End Plates", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol.15, No.2, June 1982, pp.82-92.

8. S.A. Mahin, and V.V. Bertero, "Problems in Establishing and Predicting Ductility in Structural Design", Proceedings of the International Symposium on Earthquake Structural Engineering, St. Louis, Mo., Vol.1, 1976, pp.613-628.

9. P.J. Moss, A.J. Carr, and A.H. Buchanan, "Seismic Response of Low Rise Buildings", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol.19, No.3, September 1986, pp. 180-199.

10. K.J. Thompson, and R. Park, "Seismic Response of Partially Prestressed

Concrete Systems", Journal of Structural Division, American Society of Civil Engineers, Vo.106, ST6, 1980, pp.1755-1775.

11. K. Kitayama, S. Otani, and H. Aoyama, "Earthquake Resistant Design Criteria for Reinforced Concrete Interior Beam-Column Joints", Proceedings of Pacific Conference on Earthquake Engineering, Vol.1, Wairakei, 1987, pp.315-326.

12. R.D. Hanson, and N.H. McClamroch, "Pseudodynamic Test Method for Inelastic Building Response", Proceedings of 8th World Conference on Earthquake Engineering, Vol.6, San Francisco, 1984, pp. 127-134.

13. S. Mahin, and V.V. Bertero, "An Evaluation of Inelastic Seismic Design Spectra", Journal of Structural Division, American Society of Civil Engineers, Vol.107, ST9, 1981, pp.1777-1795.

14. T. Iwasaki, K. Kawashima, K. Hasegawa, T. Koyama, and T. Yoshida, "Effect of Number of Loading Cycles and Loading Velocity on Reinforced Concrete Bridge Piers", 19th Joint Meeting US-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, 1987.

15. "Code of Practice for General Structural Design and Design Loadings for Buildings (NZS 4203:1984)", Standards Association of New Zealand, 1984.

16. L.S. Johal, D.W. Musser, and W.G. Corley, "Influence of Transverse Reinforcement on Seismic Performance of Columns", Proceedings of 3rd US National Conference on Earthquake Engineering, Vol.2, Charleston, 1976, pp.1227-1237.

17. Minutes of the Second US-New Zealand-Japan-China Seminar on Earthquake Resistant Design of Ductile Reinforced Concrete Frames with Emphasis on Reinforced Concrete Beam-Column Connections, Tokyo, 1985.

18. Songchao Zhu and J.O. Jirsa, "A Study of Bond Deterioration in Reinforced Concrete Beam-Column Joints," PMFSEL Report No. 83-1, Department of Civil Engineering, University of Texas at Austin, 1983.

19. Draft "General Structural Design and Design Loadings for Buildings (DZ 4203)", Standards Association of New Zealand, draft currently under consideration.

NOTATION

h = overall depth of a member

ℓ_p = equivalent length of a plastic hinge in a flexural member

Δ_{max} = maximum displacement

Δ_y = yield displacement

θ_{max} = maximum rotation at a plastic hinge

θ_y = rotation in the plastic hinge at yield

ϕ_{max} = maximum curvature at a section

ϕ_y = curvature at a section at yield

μ = displacement ductility factor = Δ_{max}/Δ_y

μ_a = available displacement ductility factor