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BEHAVIOR OF STRUCTURES DURING EARTHQUAKES

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Figure 1 shows an idealized load-deformation relationship of a structural member, where P is load and * is deformation. The member can be a beam or be axially loaded; in the case of a beam the deformation would be a characteristic deflection such as the deflection at midspan; in the case of an axially loaded member the deformation would be the change in length. If load is applied to the member up to a point between A and B or between A and J, the member is deformed; when the load is released, the member returns to its original shape; i.e. the deformation becomes zero. This behavior is by definition *elastic*.

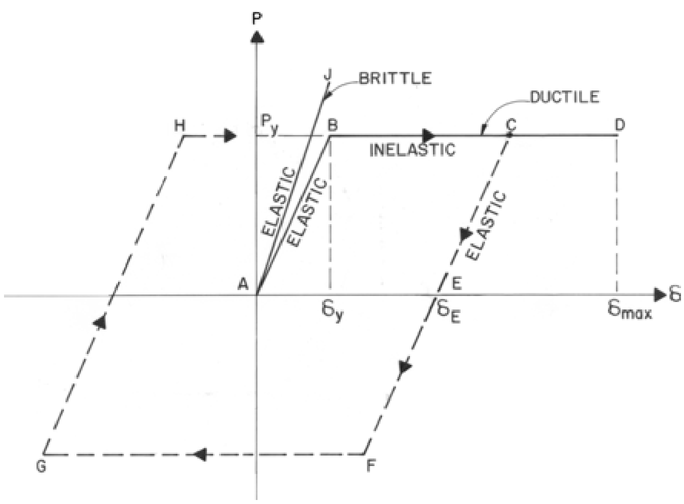

Figure 1 Idealized load-deformation relationship of a structural member.

On the path A-B-D, Point B is called the *yield point*; beyond the yield point the member is deformed permanently. Permanent deformation is by definition *plastic* or *inelastic*. If the load is maintained at the yield level, P_y, until the deformation at C is reached, and then the load is removed, the load and deformation follow along the path C-E, which is an elastic path, and at zero load there is a permanent residual deformation, $*_E$. A structural member that follows the path A-B-C-E when loaded is called *ductile* and its behavior is called *ductile*. If a member does not have ductility, it will fail at its elastic limit, such as at Point J on path A-J. Such a member and its behavior is called *brittle*.

When a member is designed for gravity or wind load, the ultimate load level (the working load times a load or safety factor), is always designed to be in the elastic range (at or below Point B or Point J). For earthquake loading on the other hand, the critical elements must be ductile since the member is designed to go into the inelastic range between points B and D. Further, during an earthquake, the member will vibrate through many cycles following a path such as A-B-C-F-G-H. The paths of succeeding cycles will not be identical, since the member will degrade due to the accumulating damage of each cycle. Earthquake loading is not limited to the elastic range, since the structural costs would be prohibitive in relation to the probability of occurrence of the design earthquake.

The forces in a structure due to an earthquake are *displacement* driven: the ground moves both horizontally and vertically, dragging the structure with it. The various masses within the structure resist the movement due to their inertia, giving rise to forces called inertial forces or seismic forces. If the structure were designed to remain elastic, the inertial forces developed due to the design earthquake would be the maximum forces the earthquake could impose on the structure; these forces are called the elastic level forces. If the structure is designed to be ductile and go into the inelastic range, the inertial forces will be limited by the yielding of the various critical members; i.e. the forces in the members cannot go higher than their maximum inelastic strengths (the yield point in the ideal relationship of Figure 1). Another way of looking at it is to think of the yielding of the various members acting as fuses to limit the inertial forces in the structure.

Figure 2 shows plots of the base shear (the total horizontal inertial force on the structure) versus a characteristic horizontal deflection (such as that at the roof level) for a ductile structure, a structure with low ductility, and a brittle structure.

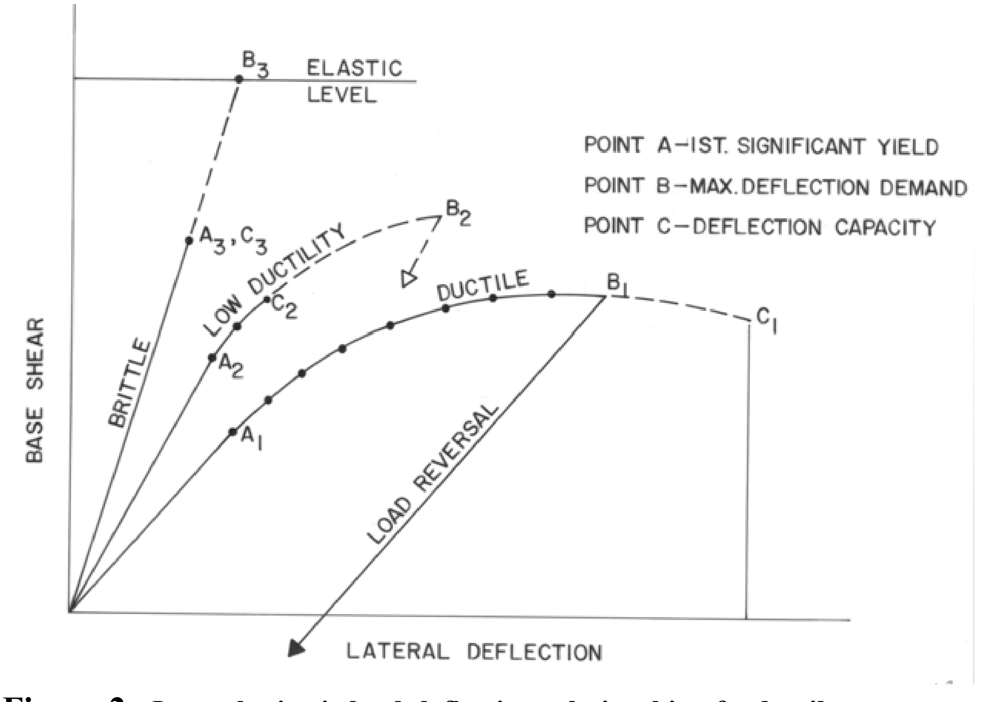

Figure 2 **Lateral seismic load-deflection relationship of a ductile structure, a structure with low ductility, and a brittle structure.**

With a brittle structure, the only way for the structure to survive, is for the elastic limit/maximum strength of the structure (Point A_3) to be higher than the elastic level (higher than Point B_3); otherwise the inertial forces will rise until the critical members fracture.

For a ductile structure, Point A_1 is where the first significant yielding occurs, and the succeeding points on the curve from A_1 to B_1 represent yielding at other locations. The structure will survive if it has sufficient ductility, i.e. if the maximum deflection demand (Point B_1) is less than the deflection capacity of the structure (Point C_1). The deflection capacity is a function of the ability of the critical members to deform sufficiently in the inelastic range without fracturing; i.e. to have sufficient ductility. A structure with low ductility will not survive if the deflection demand (Point B_2) is greater than the deflection capacity (Point C_2).

Proper detailing of structural members and their connections is essential for the necessary ductility. The extent of the required detailing can only be determined by dynamic testing of components or systems, and only the details of components and systems that have been so tested should be relied upon for earthquake resistance.

The inelastic action absorbs a substantial portion of the energy imparted to the structure by the earthquake. In Figure 2, the area within the ductile structure curve (above the horizontal axis), from the origin to A_1 to B_1 to the intercept of the load reversal path with the horizontal axis, equals the energy absorbed by plastic deformation for that half cycle of vibration. By contrast, for a structure designed to be elastic, Points A_3 and C_3 would coincide with or be higher than point B_3 on the brittle structure “curve” of Figure 2; the area within the brittle structure "curve," from the origin to B_3 and its return to the origin along the same path, is zero.

A second effect that absorbs energy is damping, which resists motion due to internal friction between parts at connections and within the material of the members themselves. The resistance of, and the friction within non-structural elements also contributes to damping. Without inelastic action or damping, a structure would vibrate indefinitely.

In current structural engineering practice, most structures are designed for earthquake at the level of first significant yielding (Point A_1 in Figure 2) using equivalent static forces (to represent the inertial forces in the structure at that level) and assuming elastic behavior. The inelastic forces and deformations are not calculated; thus, the peak inelastic base shear and the maximum deflection are not known.

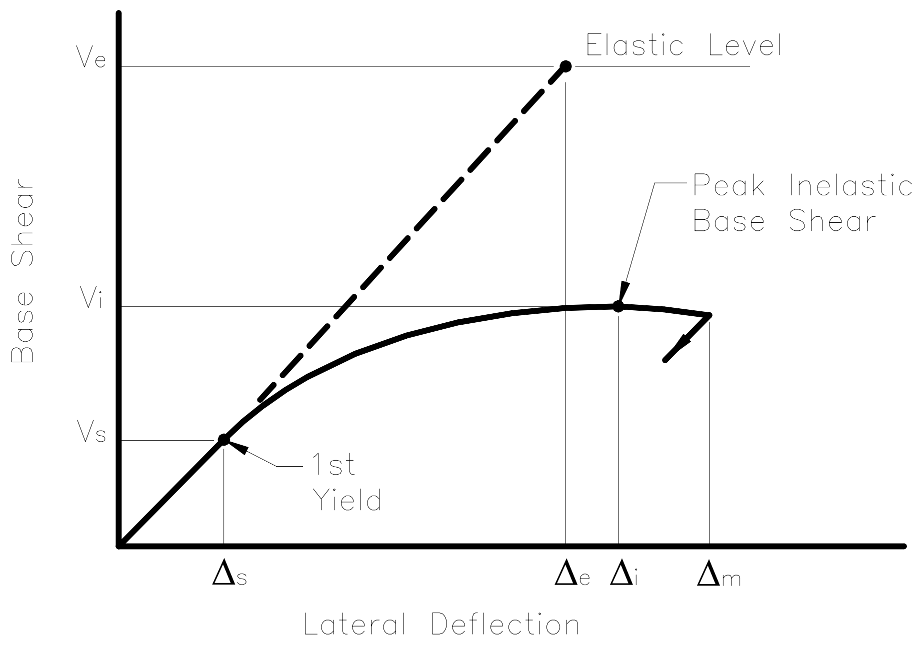

Figure 3 Inelastic behavior of ductile structure during design earthquake.

The level of first significant yielding and the peak inelastic base shear are designated *a priori*, by modifying the elastic level base shear (which is specified in codes) with empirical factors called the *response modification factor*, R, and the *overstrength factor*, S, respectively. R and S are generally based on judgement, and depend on observed behavior of various structural systems during earthquakes.

Figure 3 shows the base shears and displacements for a ductile structure subjected to the design earthquake. The equations for the base shears at first significant yielding and at the peak inelastic strength are as follows:

$$\boldsymbol{V_s = C_s W = \frac{C_e}{R} W} \qquad \text{EQ-1}$$

$$\boldsymbol{V_i = C_i W = C_s W \Omega = C_e \frac{\Omega}{R} W} \qquad \text{EQ-2}$$

where V_s is the seismic base shear at first significant yielding, V_i is the peak inelastic base shear, V_e is the elastic level base shear, W is the weight of the structure, C_s is the seismic base shear coefficient at first significant yielding, C_i is the peak inelastic base shear coefficient, and C_e is the elastic level base shear coefficient.

The terms *response modification factor* and *overstrength factor* are misleading. The response is not being modified *per se*; the *response modification factor* is a design factor that determines the first significant yielding of the structure. There is no overstrength *per se*; if the inelastic base shear was not greater than the base shear at first yield, the base shear at first yield would have to be set at a higher level to obtain good behavior (i.e. R would have to have a lower value).

The best approach to seismic design is to identify which "actions" (e.g., end moments in beams or axial forces in diagonal braces) will behave inelastically, and to design and detail the affected members and components so that these inelastic actions will have sufficient ductility to rotate or displace as the earthquake demands, without failure. All other actions need to be designed elastically for the forces that can be imposed on them by the inelastic actions and the gravity loads. In current practice, since the internal inelastic forces are unknown, members or elements which can exhibit brittle behavior such as steel column buckling, or partial penetration groove welds fracturing when subject to tension, are conservatively designed by multiplying the forces calculated at first significant yield by S.

Ductility demand is indirectly controlled in codes by prescribed limits on inelastic story drift (ratio of inelastic displacement to story height). Estimated inelastic story drift is calculated by multiplying the story drift calculated at first significant yielding by an *a priori* deflection amplification factor (called C_d in codes). The drift limits are intended to limit the structural and architectural damage due to an earthquake. The drift limits vary by code, but examples are 1% for emergency facilities that need to be operational after an earthquake, 1.5% for buildings with a large number of occupants, and 2% for all other buildings. For buildings with masonry shear walls, the limits are lower: 0.7% or 1.0%. It may be that other structural systems should have specific drift limits as well. Unlike deflection criteria for live load or wind design, which are for serviceability, deflection control for earthquake is a safety criterion and should not be ignored by the designer.

Forensic Engineering in Construction® is published by Zallen Engineering, 1101 Worcester Road, Framingham, MA 01701. Comments are welcome.
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