3.1 – FAILURE CRITERIA (LIMIT STATES)

by V. Bertero* and B. Bresler*

INTRODUCTION

Aseismic design is only one aspect of the design process. In this process, the designer must establish functional and environmental demand conditions on a building and acceptable levels of performance under these conditions. In terms of aseismic design, this requirement calls for establishing critical design earthquake or earthquakes and corresponding acceptable levels of performance or failure criteria. Usually, this problem is stated in terms of establishing design loads and their critical combinations and in terms of permissible limits of structural response under these loading conditions.

The establishment of appropriate loadings and their critical combinations requires decisions as to failure criteria and is the most difficult problem in the design process. One of the major difficulties in establishing such loadings and combinations is the uncertainty associated with predicting future ground motions and that associated with the complex behavior of soil-building systems under severe ground motions. An additional problem is caused by socio-economic requirements for greatest safety at a least reasonable cost. In order to optimize a design or to maximize utility [1], an estimate of economic losses resulting from failure is required. The term failure as used herein is synonymous with "inadmissible limit states" and includes all modes of undesirable behavior, from damage to cosmetic appearance to collapse, which may render buildings unfit for use [1].

OBJECTIVES AND SCOPE. - Other contributions to the Panel on Design and Engineering Decisions will deal with problems of optimization, consequences of failure, and codes. Therefore, the main objective of this paper is to discuss the failure criteria (inadmissible limit states) which should be considered in aseismic design of buildings. After discussing the principal failure criteria (serviceability and ultimate limit states) presently used in design, results from surveys and analyses of building damage during recent earthquakes are briefly reviewed. These recent observations indicate that an additional category of limit states related to damage which cannot be properly assigned to either serviceability failure or inadmissible ultimate limit states is needed. A discussion of damageability criteria and possible forms of damageability indices is included. Observations of damage in recent earthquakes have clearly indicated that a significant number of existing buildings are hazardous and may suffer varying degrees of damage even under moderate earthquakes. The cumulative effects of aging and other sources of possible distress--such as extreme climatic environment, wind, and fire--must therefore be considered in designing new buildings and in evaluating hazards in existing buildings.

DESIGN BASED ON LIMIT STATES

DEFINITION OF LIMIT STATES. - All structures must be designed to sustain safely all loads and deformations liable to occur during construction and in use, and to have adequate durability during its service life. A structure, or a part of a structure, is rendered unfit for use when it reaches

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a particular state, called a "limit state," in which it ceases to fulfill the function or to satisfy the conditions for which it was designed [2]. To define the different limit states, it is necessary to identify the various events that might lead to some cost of "disutility" to the occupant, owner, or designer. The different limit states are presently grouped as either serviceability or ultimate limit states. The events normally considered in limit state design and the applications of limit state philosophy to practical design methods are discussed in Refs. 2, 3, and 4. The format used in formulating the limit state design philosophy encourages the use of probabilistic methods where sufficient statistical information is available [3,4]. Because of uncertainties involved in defining the the design earthquakes, as well as the structural parameters controlling the mechanical behavior of a building, a probabilistically formulated limit state design philosophy is well-suited for developing aseismic design methods. A logical approach to the aseismic design of a structure is that of comprehensive design.

COMPREHENSIVE DESIGN. - Sawyer [5] discussed a comprehensive design procedure in which the resistance of the structure to the various failure stages is correlated with the probability of the corresponding excitations, so that the total cost, including the first cost and the expected losses from all the limit stages, is minimized. Failure of a structure under increasing loads generally occurs in successively more severe stages under successively less probable levels of load. To illustrate this point, the relationship shown in Fig. 1 shows the failure stages versus a monotonically increasing pseudo-static load for a typical statically indeterminate reinforced concrete building. Due to the variabilityof loss for a given load (or the variability of load for a given loss), the relationship shown in Fig. 1 should be considered as representing mean values of the random variables involved. The full redistribution, as shown in Fig. 2, can, in some cases, involve large variances [6].

In comprehensive design, identification of the potential modes of failure requires prediction of the mechanical behavior of a structure at each significant level of critical combinations of all possible excitations to which the structure may be subjected. Because it is usually not possible to consider real behavior under the actual critical excitations to which the structure may be subjected, it is common to base structural design on idealized conceptions of mechanical behavior under a simplified set of excitations. The sources, treatment, and effects of the different types of excitations which may be exerted on structures are summarized in Fig. 3 [7]. The sequence of actions to which a structure may be subjected often consists of unpredictable fluctuations in the magnitude, direction and/or position of each of the individual excitations. The only characteristics that may be estimated accurately are the extreme values between which each of these actions will oscillate. These types of actions have been classifed in Fig. 3 as generalized or variable-repeated excitations.

The particular phenomena associated with variable-repeated excitations are classifed as long-endurance fatigue; low-cycle fatigue; and incremental collapse. Long-endurance fatigue is a critical consideration only in special structures. A review of results regarding low-cycle fatigue, which is associated with repeated-reversible actions, indicates that the real danger of these actions is not fracture of the structural material, but deterioration of the stiffness, particularly in the case of reinforced concrete [7]. Incremental collapse is associated with progressive development of excessive deflections which occur under the cyclic applications of different combinations of peak actions. Because deterioration of stiffness can lead to an undesirable increase in deformations, in examining actual generalized excitations, the effects of alternating excitations cannot be treated independently, as is usually done, from those caused by excitation patterns leading to incremental deformations [7]

CURRENT FAILURE CRITERIA IN ASEISMIC DESIGN

GENERAL GOALS AND CURRENT PRACTICE. - The general philosophy of earthquake-resistant design for buildings other than essential facilities has been well-established and proposed to: (1) prevent nonstructural damage in minor earthquake ground shakings which may frequently occur in the service life of the structure, (2) prevent structural damage and minimize nonstructural damage in moderate earthquake shakings which may occasionally occur, and (3) avoid collapse or serious damage in major earthquake ground shakings which may rarely occur. This philosophy is in complete accordance with the concept of comprehensive design. Current design methodologies, however, fall short of realizing the objectives of this general philosophy. Application of the comprehensive design approach to aseismic design would entail replacing the load and load probability scales by the seismic excitation intensity and intensity probability scales, respectively (Figs. 1 and 2). Practical application of this approach is, however, considerably more complex because of difficulties involved in assessing the relationship between loss and seismic excitation. According to the concept of comprehensive design, the ideal design is that which results in the minimum total cost, including possible losses, for all limit states. However, this ideal is not an immediate practical possibility in actual design. No practical design method has yet been developed that satisfies simultaneously all the requirements imposed by the different limit states. In practice, the most critical limit state is used as the basis for proportioning members in the preliminary design; all other main limit states should then be checked through a comprehensive analysis. The advantages of developing a design method based on two failure stages have been discussed by Sawyer [5], and a design method based on two behavioral criteria (collapse and loss of serviceability) and on four optimizing criteria has been developed [8]. Application of this method to the aseismic design of ductile moment-resisting frames seems feasible and practical [9].

Because current design practice in regions of high seismic risk focusses on collapse of the main structure as the controlling limit state, the resulting design must be checked for serviceability requirements under normal loading conditions. Examination of building damage resulting from recent severe seismic ground shaking reveals that although buildings were far from reaching the collapse limit state, the degree of nonstructural damage was so great as to constitute failure. Therefore, it is necessary to introduce a new group of limit states based on damageability. Before discussing this need in more detail, the failure criteria used in present aseismic design practice should be considered.

SERVICEABILITY REQUIREMENTS. - Although the conditions leading to serviceability limit states under normal loading have been defined in general terms [2], specific quantitative limits have not been adequately determined. More practical and consistent quantifications are needed for determining failure stages of structural and nonstructural components under all types of service excitations. For example, it has been recommended that the maximum tolerable drift index for walls be limited to 0.002 [10]. On the other hand, in the case of seismic loads, the 1976 Uniform Building Code (UBC) specifies a maximum index of 0.005. Since seismic forces specified in this code apply to designs at service load levels, the UBC value for seismic drift appears to be unconservative when compared to that suggested in Ref. 10.

In quantifying the serviceability limit states for seismic excitations, it is necessary to determine the building's function and the level of excitation intensity under which the facility should remain serviceable. In the case of essential facilities, these should not only be safe, but they should be functional for emergency purposes even after the occurrence of the maximum credible excitations expected during the service life of the building. Some quantitative limits for serviceability requirements for essential facilities are shown in Fig. 4. Althoug the seismic design forces for the different codes considered in this figure are not strictly comparable, the significant differences between these specified tolerable drift indices indicate the need for more thoroughly investigating the degree of damage constituting failure and corresponding tolerable drift criteria.

ULTIMATE OR SAFETY REQUIREMENTS. - Analysis of the causes leading to ultimate failure of the building reveals that this can be induced by different failure mechanisms acting independently or in combination. Some of these limit states appear to be extremely critical under pseudo-static loads, while they may be negligible under dynamic loads. Under a sustained pseudo-static overload, for example, the limit state caused by transformation of the structure into a mechanism lead to instability of the whole structure; this is usually not so under dynamic loading. Actually, present aseismic design methods are based on the assumption that large displacements (large ductility) develop after the structure is transformed into a mechanism. The distinction between pseudo-static and dynamic effects also applies in the case of ultimate limit state caused by deformation instability.

Failures under Generalized Dynamic Excitations. - Collapse of a structure can occur as a consequence of "low-cycle fatigue" or "incremental deformations" under excitation intensities lower than those required to induce instantaneous collapse if these excitations are considered as monotonically increasing. As pointed out in Refs. 1 and 7, cumulative damage resulting from a long, strong ground motion, a short main shock followed by a succession of aftershocks, or a combination of the main shock and another consequential event or environmental exposure such as fire, can lead to either one of the above two phenomena and therefore merits considerably more attention that it has received.

Yamada and Kawamura [11] have discussed an ultimate aseismic design philosophy of reinforced concrete based on low-cycle fatigue. This type of failure is very sensitive to detailing and quality control of materials and workmanship used in construction. If errors in design or construction, or lack of quality control of materials and of workmanship are eliminated, then application of adequate seismic design provisions with possible further improvements [12], will result in structural designs in which low-cycle fatigue would not control the design. By detailing the expected critical regions of different structural members according to recently proposed seismic code provisions, the energy absorption and energy dissipation capacity developed under cyclic reversals of deformation will be so large as to resist the energy input of even the toughest of credible seismic motions. Even under the most severe ground motions recorded, the number of reversals that can occur between opposite peak deformations having the maximum intensity is not usually large enough to be of serious concern [12]. It should also be noted that under full reversals of symmetrically yielding and strain-hardening or strain-softening structures, the P-Δ effect is cancelled out (Fig. 5).

Studies carried out at Berkeley [13] have shown that one case where low-cycle fatigue could control the design involves members that are used as structural dampers to dissipate energy. One typical example of such a case is that involving coupling girders in coupled wall systems [13]. However, failure of these members does not necessarily lead to complete structural failure. Since these elements act as safety fuses between two different structural resistant systems, their failure would lead to a change in the dynamic characteristics of the system rather than to a brittle failure of the complete system.

A schematic illustration of the <u>incremental collapse</u>, denoted as "crawling collapse," is shown in Fig. 6. Recent studies [14] have shown that this type of failure can control the aseismic design of structure, particularly at sites near the source of seismic ground motions containing severe, long acceleration pulses. For example, the study of the response of a multistory steel frame, optimally designed using a nonlinear method, to seismic ground motions derived from those recorded during the 1971 San Fernando earthquake shows that the frame will collapse due to the type of incremental deformations illustrated by the first story displacement time-history response of Fig. 7. The danger of incremental collapse is aggravated by the high probability that several aftershocks of intensities and dynamic characteristics comparable to that of the main shock will occur. As Newmark and Rosenblueth [1] have pointed out, it is not unusual for a structure which is able to withstand a major shock with visible damage, to collapse during an aftershock.

Although the P-Δ effect is not a factor in failures due to low-cycle fatigue, it is of paramount importance in failures of an incremental collapse type. As a structure is deflected away from its original vertical equilibrium position, the increment in sidesway deflection under repetition of the same acceleration pulse will increase since the structure's available net yielding resistance against lateral inertial forces is considerably reduced by the P-Δ effect (Fig. 6). Accumulation of these increasing incremental deflections can lead to an instability phenomenon under a working load combination (gravity forces plus wind or minor earthquake). Figure 6 indicates that structural instability under working loads may be prevented or delayed by a reduction in the maximum tolerable story drift, by an increase in the yielding strength against lateral forces, or by a combination of these two possibilities. It should be noted, however, that the only advantage in increasing the initial stiffness without either modifying the yielding strength or maximum tolerable story drift will be a small increase in the energy absorption and energy dissipation capacity. Such an increase is illustrated in Fig. 8(a). This figure also indicates that an increase in initial stiffness without a reduction in tolerable story drift will lead to a considerable increase in ductility demands, and, therefore greater structural damage. A reduction in the acceptable story displacement ductility will generally lower the danger of instability because such a reduction implies an increase in the required yielding strength of the structure which in turn usually requires a corresponding increase in the intial stiffness. The end result is a story drift at yielding equal to or less than that corresponding to a structure with a lower yielding strength, and a considerably smaller story drift at ultimate condition.

The behavior depicted in Fig. 6 suggests the approximate design method, illustrated in Fig. 8, for preventing or delaying the deformation instability under working load levels. The method is based on the assumption that maximum tolerable story drift, Δ_s^{MAX}, and story shear due to lateral working loads, S_s^W, are known. The total axial force acting on a story during severe seismic shaking is also assumed to be known since it depends only on the gravity forces acting above that story, P_s^G. Two different examples of possible inelastic behavior are considered in Fig. 8. If the mechanism deformation is of a perfectly plastic type, it will be sufficient to draw a line, BO', parallel to OA through point B [Fig. 8(a)]. If the mechanism deformation of the structural system is developed with some strain-hardening, it will be necessary first to locate point B'. Then drawing B'O' with a slope equal to the expected rate of strain-hardening, intersection O' will give the mechanism yielding strength required, S_s^Y, as shown in Fig. 8(b). Comparison of Figs. 8(a) and 8(b) illustrates the advantage of having a structural system whose mechanism deforms with some strain-hardening.

Experimental results [15] have shown that requirements for preventing instability of structural members depends on the desired level of ductility. The larger the tolerable ductility, the more stringent the requirements should be. Under loading reversals, when the ductility value exceeds a certain limit, there is a sudden drop in resistance against instability, particularly in the case of reinforced concrete structures.

DAMAGEABILITY LIMIT STATES

LESSONS LEARNED FROM RECENT EARTHQUAKE DAMAGES. - Review of recent earthquake damage reveals that many buildings which did not collapse had to be either completely or partially demolished due to the high amount of nonstructural and structural damage which constituted failure. Numerous buildings whose structural systems did not undergo any significant structural damage, suffered such damage to nonstructural components as to render the entire building unfit for use. As previously pointed out, most present aseismic design methods focus on collapse (ultimate strength and displacement ductility) of the main structural system as the essential limit state. The main problem in applying such methods is in establishing the proper displacement ductility value. Selection of just one value cannot ensure that a structure will be safe and economical or that damage will remain within acceptable limits in all cases.

Although it is generally recognized that the most important single cause of damage is deformation, the types of deformations primarily responsible for damage to nonstructural components remain unclear. It has been argued that while lateral displacement ductility factors generally provide a good indication of structural damage, they do not adequately reflect damage to nonstructural elements [16]. Nonstructural damage is more dependent on the relative displacements (interstory drift) than on the overall lateral displacements. Aseismic design methods must incorporate drift (damage) control in addition to lateral displacement ductility as design constraints. Story drifts and drift ductility factors may also be useful in providing information on the distribution of structural damage, although conventionally computed story drifts are unreliable indicators of potential structural or nonstructural damage to multistory buildings. In some structures, a substantial portion of horizontal displacements results from axial deformations in columns. Story drifts due to these deformations are not usually a source of damage [Fig. 9(a)]. A better index of both structural and nonstructural damage, particularly for frames tightly infilled with partitions, is the tangential story drift index R. As schematically indicated in Fig. 9(b), this index is used to measure the shearing distortion within a story. For the displacement components shown in Fig. 9(c), the average tangential drift index is equal to $R = (u_3 - u_1)/H + (u_6 + u_8 - u_2 - u_4)/2L$.

Glogau [17] discussed the different types of deformations that could cause damage to nonstructural elements as well as formulated different damage control strategies. Broad damage mitigation strategies have also been discussed by Kost and associates [18].

DAMAGEABILITY. - Establishment of a proper failure criterion based on damageability requires development of a methodology for damageabilility as an inadmissible limit state under extreme (potentially catastrophic) environmental hazards of the whole building rather than that of the bare structure. Similar to other failure criteria for aseismic design, damageability limit states depend on the type of ground motions being generated. Not only should the intensity of these excitations be considered, but their general dynamic characteristics and their combinations with loads resulting from gravity forces and environmental effects should be accounted for. Damageability limit states can be considered as a category that bridges the gap between serviceability and safety against collapse. Although the primary causes of damageability with which we will be concerned are due to significant overexcitations (large deformational behavior of structural and nonstructural components), effects of service excitations on damageability should not be ruled out. Inadmissible limit states are usually described in terms of limiting the levels of structural response, e.g. maximum displacement, crack width, forces and moments. Although such structural responses may be related to the risk of life-loss, injury, and to economic losses resulting from damage, the relationship of structural response to damage and to socio-economic losses has not been clearly established. To facilitate the establishment of such a relationship, it is proposed to define indices of damageability for a given load or environment exposure history which can be used an an indicator of a limit state condition.

<u>DAMAGEABILITY CRITERIA</u>. - In considering damageability, three general types of damage must be distinguished: (1) <u>local damage</u> - limited to one or several typical elements; (2) <u>global damage</u> - overall damage in a particular event related to the total building; and (3) <u>cumulative damage</u> - overall damage resulting from a series of events, such as strong earthquakes followed by a series of aftershocks, or by other consequential or independent events such as fire, or some other combination of normal and catastrophic events.

Physical damage to both structural and nonstructural components is related to structural response characteristics. Recent advances in methods of structural analysis for complex nonlinear behavior under a variety of dynamic load conditions as well as under fire [19-21] and other environmental exposures provide a basis for investigating damageability. One problem encountered in these investigations involves the proper modeling of nonstructural components to study their interaction with structural models. Because there is no reliable data on the actual mechanical behavior of these components, it will be necessary to study the type and amount of deformation and/or forces that are required to produce different levels of damage in masonry, wood panels, gypsum boards, glazed openings, equipment, etc. Another difficult in realistically assessing structural response and potential damage in existing structures subjected to earthquake is in properly evaluating the current state of the building at the time of the earthquake. Such evaluation involves considering the effects of (1) previous exposure to climatic environment (thermal changes or shrinkage), causing a state of residual stress or distress, and deterioration in structures due to aging and corrosion; (2) degradation in strength and stiffness caused by previous exposure to high winds, fires and/or earthquakes; (3) other disturbances or movements of the foundation; and (4) changes in strength and stiffness due to alterations, repair, or strengthening. Because any one of these conditions can significantly alter structural response, one of the problems that must be included in the study of damageability is the effects of variations in load and environmental histories, and the residual conditions in the structure (residual stresse, cracking, corrosion, and other changes in stiffness or strength of the materials). Once the "present state" of a building has been properly assessed, and the mechanical (or mathematical) model is clearly described in terms of the intensity and characteristics of the ground motion, the response of a building (structural and nonstructural components) can be determined. A general evaluation framework, which is based on a sequence of basic procedures starting with the simplest models and employing more complex models as needed to achieve desired reliability, has been formulated [22]. This procedure is referred to as "screening."

Several procedures for evaluting earthquake safety of existing buildings were proposed following the 1971 San Fernando earthquake and have since been incorporated into practice [23]. These methods fall into two general categories. The first includes procedures which may be found in mandatory regulations; the second, proposals which focus on methodology and are published as technical reports or papers. These methods did not, however, address the problem of global or cumulative damage, nor did they provide a means for including nonstructural damage in an overall assessment of damageability.

DAMAGEABILITY INDICES. - An index of local damageability, D_i, for a given element i in a building exposed to a specified load or environmental exposure is defined here as the ratio of building response demand for this element (d_i) to its corresponding resistance capacity (c_i) that is, $D_i = d_i/c_i$, where capacity c_i is the limit value for building response without damage. Both structural and nonstructural elements should be considered in evaluating damageability index D_i. For the design of new buildings, values of d_i and c_i must consider randomness in loading demand as well as in "as built" condition determined by quality control during construction. With properly defined values of d_i and c_i, damage will occur when $D_i > 1$; when $D_i < 1$, no local damage should occur, and in this case, D_i should be assigned a value of zero.

Overall or global damageability index D_g may be defined as the sum of nonzero values of D_i, including structural and nonstructural components which might be damaged in a particular event of extreme exposure. Values of D_i must be weighted by an appropriate importance (life hazard, cost, etc.) factor p_i, as $D_g = \sum_n p_i D_i$. The sum is taken over n damageable elements, including both structural and nonstructural components. Index D_g should be normalized to $\overline{D}_g$ in order to use the latter for comparing two buildings or two alternate designs of the same building. Several possible ways to accomplish this normalization should be explored. For example, $\overline{D}_g$ may be defined as $\overline{D}_g = \overline{D}_g/\sum_n p_i$, or, more appropriately, as $\overline{D}_g = D_g/\sum_m p_i$, where n is the number of damageable elements, m is the total number of elements (both damageable and nondamageable), and $\sum_m p_i$ reflects some overall current value of a building.

The cumulative damageability index, D_c, may be defined as the sum of nonzero values of $p_i D_i$, including structural and nonstructural components which might be damaged as a result of a specified sequence of events, for example, fire exposure, repair of fire damage, strong earthquake, with specified strong aftershocks. Such factors can be taken into account in evaluating local damageability by introducing service history influence coefficients η_i (for demand) and $\varkappa_i$ (for capacity), which are also influenced by the randomness of these influences. Then $D_i' = \eta_i d_i \varkappa_i c_i$, where D_i' is the current nonzero local damageability index which accounts for the assumed service history of a building. If N is the number of damageable components in such a case, the $D_c = \sum_n p_i D_i'$. Normalized value $\overline{D}_c$ can then be expressed as $\overline{D}_c = \overline{D}_c/\sum_m p_i$. For old buildings, evalution of the damageability index is further complicated by the significant influence the service history of a building may have on the values of both demand and capacity (either increasing or decreasing these values), due to such factors as aging, change in use or occupancy or in socio-economic conditions (which would affect p_i values), structural and nonstructural modification, fire damage and repair, corrosion, etc. The same problems exist for new buildings, due to the uncertainties associated with predicting future earthquakes. Then $D_g' = \sum_n p_i' D_i'$ and $D_c' = \sum_N p_i' D_i'$, where p_i' is the current importance factor (which may differ from the factor p_i used in the original design). Normalized values $\overline{D}_g'$ and $\overline{D}_c'$ for existing buildings can be defined similarly to $\overline{D}_g$ and $\overline{D}_c$ values for new buildings. The larger the value of $\overline{D}$ or $\overline{D}'$, the greater the overall damageability index of a building. When $\overline{D}$ or $\overline{D}'$ exceeds some specified limit value, the damageability risk is too great and the building should either be redesigned or strenthened, or demolished.

DAMAGEABILITY AS FAILURE CRITERION. - The general philosophy of developing a method and criteria for assessing damageability has been presented, but the methodology for evaluating the different damageability indices are still undergoing development [23]. One of the main problems encountered in developing such methodology is in defining reliable procedures for calculting the values of d_i, c_i, p_i, η_i, κ_i, and p_i'. Quantification of damageability limit states will require extensive investigation of the mechanical behavior of nonstructural elements, or, what Kost et al. [18] have termed, EFS (enclosure, finish, and service system) components. With the findings from such studies, it will be possible to develop a conceptual model for analyzing the dynamic behavior of entire soil-structure systems. Implementation of the model in damageability limit state studies will enable guidelines for assessing failure criteria in aseismic desigg to be formulated.

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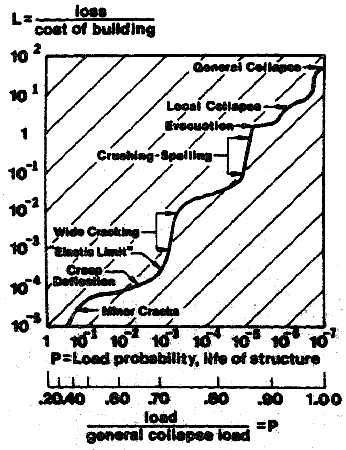

Fig. 1 Assessment of mean losses vs. load probabilities during the life of structure [5]

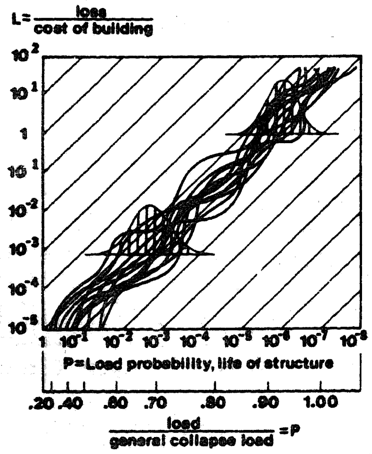

Fig. 2 Distribution of losses vs. load probabilities during the life of structure [6]

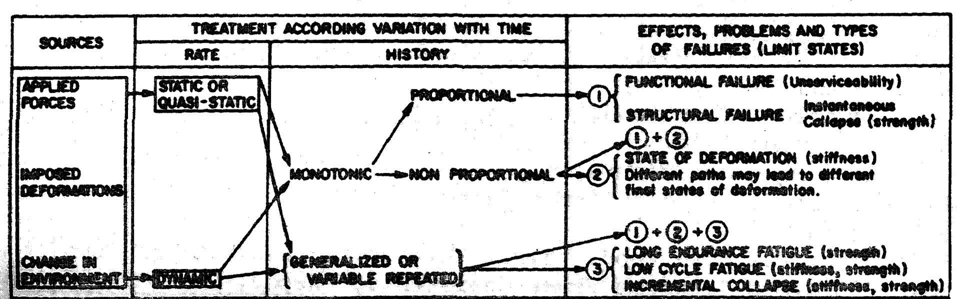

Fig. 3 Sources, treatment, and effects of excitations on structures

U.S. VETERAN ADMINISTRATION HOSPITALS(1)	1976 UBC(2)	NEW ATC-3 PROPOSAL(1)	MEXICO FEDERAL DISTRICT(1)	NEW ZEALAND(1)
0.0078 0.0026(a)	0.005 0.01(b)	0.01	0.005	0.006(c) 0.01(d)

(1) Maximum value considering inelastic deformations.

(2) Maximum value based on code prescribed forces at service level.

(a) For glazed openings.

(b) Equipment must remain in place and be functional.

(c) When nonstructural components are not separated from the structure.

(d) When nonstructural components are separated from the structure.

Fig. 4 Lateral interstory drift index limitations for essential facilities

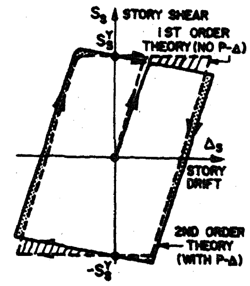

Fig. 5 Effect of P-Δ on low-cycle fatigue

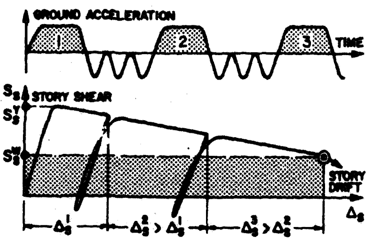

Fig. 6 Incremental collapse

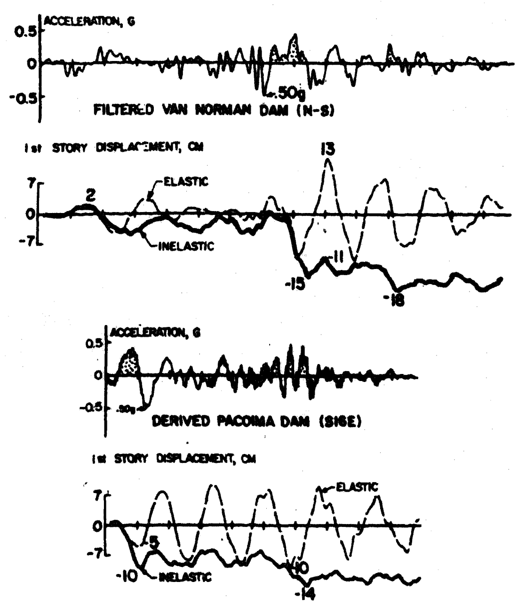

Fig. 7 First-story displacement time-history response [14]

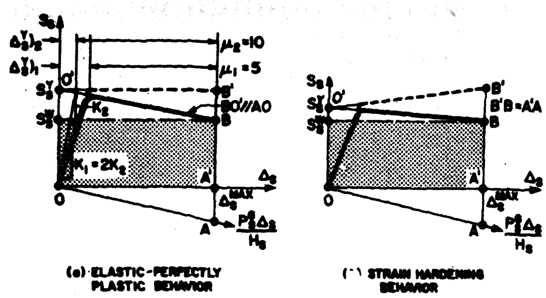

Fig. 8 Determination of required yielding strength to avoid instability due to P-Δ

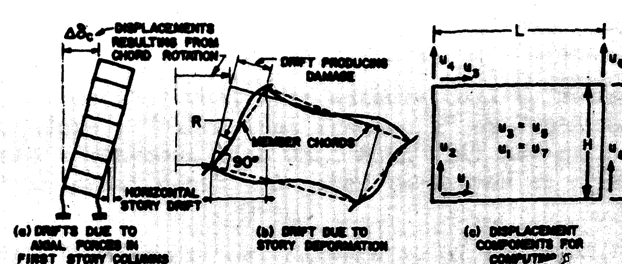

Fig. 9 Story drift

DISCUSSIONS

Gostu Venkatesulu (India)

The failure criteria evolved appears to be mainly for Buildings as non-structural damage is also considered. Failure criteria for bridges will necessarily have to be different from those of buildings. Even for bridges we have to differentiate between minor bridges and major bridges.

Perhaps for the innumerable number of minor bridges on any new road, rigorous design for earthquake forces may not be necessary. If any are minor bridge collapses, the cost may be borne. It may not be prudent to design all minor bridges for earthquake forces. Some executive decision can be taken and a cêrtain element risk taken.

Author's Closure

I fully agree with you on the failure criteria.