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Structural Design Process with Seismic Considerations

Processus de la conception des structures et considérations sismiques

Der Entwurfsprozess mit seismischen Überlegungen

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SUMMARY

This paper discusses the structural design process with particular reference to the problems encountered in seismically active regions. The basic differences with conventional design are identified and the underlying philosophy for developing seismically resistant structures is presented. An appraisal of analysis methods is given, and some difficulties in adapting elastic solutions for post-elastic behavior are pointed out. Strong emphasis is placed on the need for considering the plastic limit state in all cases. The design process is illustrated by showing an identification of the problem, a choice of concept, and the development of the necessary experimental support for eccentrically braced steel framing.

RESUME

L'article traite du processus de la conception des structures compte tenu des problèmes spéciaux rencontrés dans les régions sismiquement actives. Il précise les différences fondamentales avec la conception conventionnelle et présente la méthodologie du développement des structures résistantes aux séismes. L'article évalue les méthodes de calcul et indique quelques difficultés dans l'adaptation des solutions élastiques au comportement post-élastique. Il met en relief la nécessité de tenir compte de l'état-limite plastique dans tous les cas. Le processus de la conception est illustré par l'identification du problème, le choix d'un concept, et le développement du support expérimental nécessaire pour une ossature métallique à contreventement excentré.

ZUSAMMENFASSUNG

Diese Arbeit diskutiert den Entwurfsprozess unter dem besonderen Aspekt der Probleme bei Bauwerken in Erdbebengebieten. Die Hauptunterschiede zum konventionellen Entwurfsprozess werden herausgestellt. Die Grundgedanken beim Entwurf von Bauwerken, welche Erdbebenlasten widerstehen können, werden vorgestellt. Verschiedene Lösungsmethoden der Baustatik werden bewertet und die Schwierigkeiten aufgezeigt, um mittels linear-elastischer Methoden auf das nichtlineare Verhalten zu schliessen. Besonderes Gewicht wird auf eine sorgfältige Untersuchung der plastischen Grenzzustände für alle Lastfälle gelegt. Der Entwurfsprozess wird am Beispiel von exzentrisch ausgesteiften Stahlbauten vorgestellt. Es wird eine Beschreibung der Problematik gegeben. Ferner wird ein Entwurfskonzept aufgezeigt und das Programm der notwendigen experimentellen Untersuchungen beschrieben.

1. INTRODUCTION

In the introductory report for Theme A, Professor MacGregor discusses the nature of structural engineering and succinctly outlines the design process involved in such work. For seismic design the general approach remains the same, but significant differences in emphasis are necessary. These pertain to the need for greater involvement of the structural engineer with the conceptual solution of the structural problem as well as with concern for the ever-present uncertainty of the loading conditions. Unquestioning adherance to codes and elastic methods of analysis may result in unsatisfactory structures, leading to total collapse and huge loss of life during a severe earthquake [4].

In many respects seismic design remains an art and places a great deal of responsibility on the structural engineer. Some of the above general ideas are elaborated upon in the paper by first identifying the problem, discussing the selection of a design, and providing an evaluation of the current approach for seismic analysis and design. The newly developed eccentrically braced steel framing is then used to illustrate the seismic design process.

2. PROBLEM IDENTIFICATION AND CONCEPTUAL SOLUTION

A conceptual solution of a structural problem for resisting lateral forces requires the highest level of structural engineering talent and judgment. By studying the proposed configuration of a structure, noting the distribution of mass and the foundation conditions, a possible lateral supporting system can be conceived.

On major structures an interaction between the architect and the engineer is imperative, the earlier the better. In devising a lateral supporting system, one must think in terms of a systems approach, i.e., floor diaphragms, their attachment to the vertical supporting system, the vertical support system itself, as well as overturning and foundation problems. Anticipation of problems arising from perforation of the floor diaphragms by stair and elevator wells, mechanical equipment, telephone ducts, etc., as well as an appreciation of the capacities of slender vertical walls or braced bays, must form the basis for selecting a structural framing system. Consideration of story drift control at service loads and ample ductility during a maximum credible earthquake for a given site are imperative.

Simultaneously with the process of selecting a structural system, a decision must be made on the materials to be used. For smaller structures, fire code permitting, wooden framing is economical and has an excellent record of performance during severe earthquakes. The use of reinforced concrete or masonry for garage enclosures or of structural steel or prestressed concrete members for larger spans often is a logical solution. On the other end of the spectrum, i.e., for tall buildings, structural steel is generally preferred, although in more recent years composite frames of reinforced concrete and structural steel have been adopted in spectacular applications [5,21].The use of structural steel often is logical in construction of large factory complexes because of ease of alterations. One- and two-story warehouse and commercial buildings normally are more economically built in reinforced concrete.

For the bulk of non-residential construction in the four- to twenty-story group of buildings, there is strong competition between reinforced concrete and structural steel. At the present time, a significant number of such buildings on the West Coast are being constructed in structural steel. The choice is based primarily on cost considerations, which change rapidly. Therefore, the engineer must have current familiarity with construction costs, although admittedly the choice of a material is often based on personal preference, either of the engineer or architect.

The more a particular building deviates from the conventional, the more framing schemes must be examined before adopting a solution. The appraisal of a framing system in seismic design has a number of special aspects, which will be considered in the next section.

3. APPRAISAL AND SELECTION OF A DESIGN

In appraising and selecting a design for a highly seismic environment, several aspects of the structural problem must be carefully scrutinized. Some of these assume far greater importance than for a conventional design.

In seismic design there is always considerable uncertainty as to the loading conditions during a major earthquake. On the other hand, for reasons of economy, the safety factors for buildings are kept small. An optimized, efficient structural system for gravity loads may not be the best choice for seismic applications. Redundancy in the structural system is desirable. Concentrating lateral resistance in a few members may not result in the best earthquake-resistant structure. For example, four shear walls, with two at each end of a building, are preferable to just two equally strong end walls. Likewise, concentrating all of the lateral resistance on one bay of a multi-bay steel frame is less desirable than distributing the resistance to several bays. Fortunately, the code-writing groups are beginning to recognize the advantages of redundancy in seismic resistant construction.

Most experiments are made to small scale, and the successful ones become a presumed standard of performance. Such results are freely extrapolated to related cases, and certainly to much larger sizes. One can hardly expect the same performance from field-erected structures, and the size effect has been poorly explored. Moreover, most of the available research is on isolated members and joints. The experiments recently completed at Tsukuba, Japan, on a full-size seven-story reinforced concrete building [18] and on a six-story steel building [6,7] are notable exceptions. But even in these cases, the member sizes are modest in comparison with many modern structures. Extrapolations from the available data to large members encountered in practice should be done with a great deal of caution.

Information learned from damage caused by past earthquakes should be related as much as possible to the design being considered. Full recognition of the differences between modern and earlier construction should be made: the days of heavy concrete fireproofing of steel members and massive partitions are gone. The steel is no longer joined by rivets, which in the past completely avoided the problems of lamellar tearing. Stringent requirements often introduced immediately after a damaging earthquake gradually tend to be relaxed. Monotonic static tests are usually considered fully adequate to demonstrate a point. Experience with the behavior of tall buildings in major earthquakes is very limited. Strong trade partisianship is evident in many cases. Unfortunately, if cost-effective simplifications are accepted a few times, they become state-of-the-art and next to impossible to change. Attachment of wood diaphragms to masonry without anchors or reluctant use of continuity plates in moment-resisting beam-column steel joints may be cited as examples.

Because of the smaller factor of safety used in seismic than in conventional design, the engineer charged with an appraisal and selection of a design should be conversant with the items discussed above.

The extensive technical literature which describes the damage incurred during major earthquakes provides useful information. Some such observations are synthesized by structural engineers. The Structural Engineers Association of California has standing committees which modify a model seismic design code on a continuous basis [16]. These recommendations gradually find their way into the basic national building codes [19]. Similar comprehensive activity is carried on in the USA by the Applied Technology Council [1], as well as by the broader-based Earthquake Engineering Research Institute [4]. On the international level, the International Association for Earthquake Engineering [3] disseminates basic information in this area, principally through quadrennial world conferences.

4. SEISMIC ANALYSIS AND DESIGN

The lateral forces given in the codes [2,19], which represent the effect of an earthquake on a structure, are a gross simplification of a very complex problem. The random dynamic repeating and reversing forces that develop during a seismic event are reduced to a set of deterministic equivalent static forces for design. Only large or monumental buildings are analyzed dynamically, and such a requirement is written into law only in the Los Angeles code for buildings over 160 ft in height or for those of irregular shape [2]. Elsewhere, the dynamic analyses are performed at best only on major buildings to obtain a better idea of the structural response and also, in some instances, to reduce the code-stipulated lateral forces. If a dynamic analysis of a structure is performed, its behavior under the maximum credible earthquake can be much better understood.

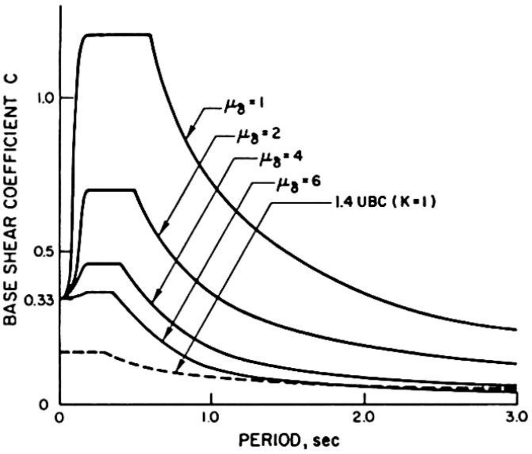


Fig. 1. Base Shear Coefficient Curves [20].

The lateral static forces specified in the codes [2,19] are much smaller than those that would be expected if a building were to respond elastically. However, because some acceptable structural damage in a major earthquake in the form of controlled inelastic or plastic deformations dissipates the input energy, dynamic analyses clearly show that forces acting on a structure are significantly reduced. This is illustrated in Fig. 1, where the behavior of single-degree of-freedom systems with different natural periods of vibration are exhibited. The base shear coefficient for an elastic system ($\mu_\delta = 1$) for this selected severe earthquake is given by the upper curve; the code values [19] multiplied by 1.4, giving approximately the threshold level at which inelastic action would begin, are given by the dashed line. Only by designing a structure capable of deforming plastically to reach a displacement six times the elastic one, i.e., for the deflection ductility $\mu_\delta = 6$, does one obtain a reconciliation between the code-specified forces and structural response.

Admittedly, the example cited is for an extraordinarily strong earthquake; for smaller quakes, one finds a less critical situation. Further, on the average, the mechanical properties of materials usually exceed their specified values, and, due to the redundancy, the loading pattern usually becomes advantageously redistributed; nevertheless, it is essential to note that in seismic design one must be assured of ductile behavior at overloads. Such behavior cannot be taken for granted.

Design engineers are well aware of the basic point made above. However, generally in the US, analyses are made using elastic concepts carried out with the aid of computers. For estimating the ductility demand of various members or connections, either for static or dynamic cases, the approach shown in Fig. 2 is often employed. From such a diagram the ductility demand μ is determined from the relationship:

$$\mu = \frac{M_o}{M_p} = \frac{\theta_o}{\theta_p} \qquad (1)$$

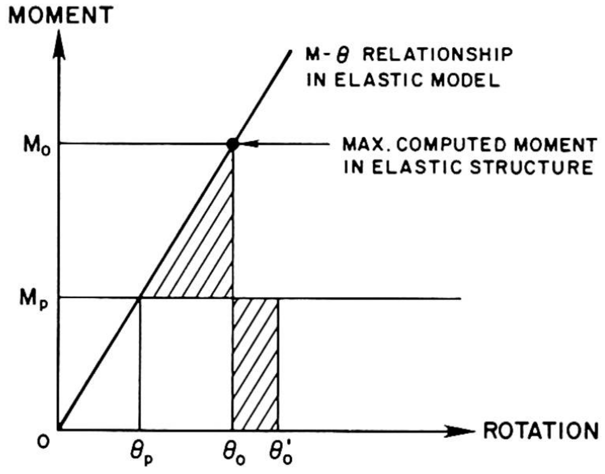


Fig. 2. Ductility Definition.

where M_o is the maximum computed member moment based on an elastic model of the structure and M_p is member plastic moment capacity. θ_o and θ_p are linearly related to M_o and M_p. A variant of the above approach consists of defining the maximum rotation θ_o' by generating the shaded rectangle to be of equal area to the shaded triangle in order to preserve equal energies for elastic and elasto-plastic cases. Unfortunately, either one of the above two schemes applies only for statically determinate cases. For example, using this approach for the three-story split K-framing system shown in Fig. 3 would be grossly in error. The relationship between the critical moments for the ultimate case (shown in Fig. 3b) to those for the elastic case (Fig. 3a) is not linearly related, nor are the rotations. For a more accurate estimation of the ductility demand of structural members and connections, elasto-plastic computer analyses of structures must be developed for use by the design engineers.

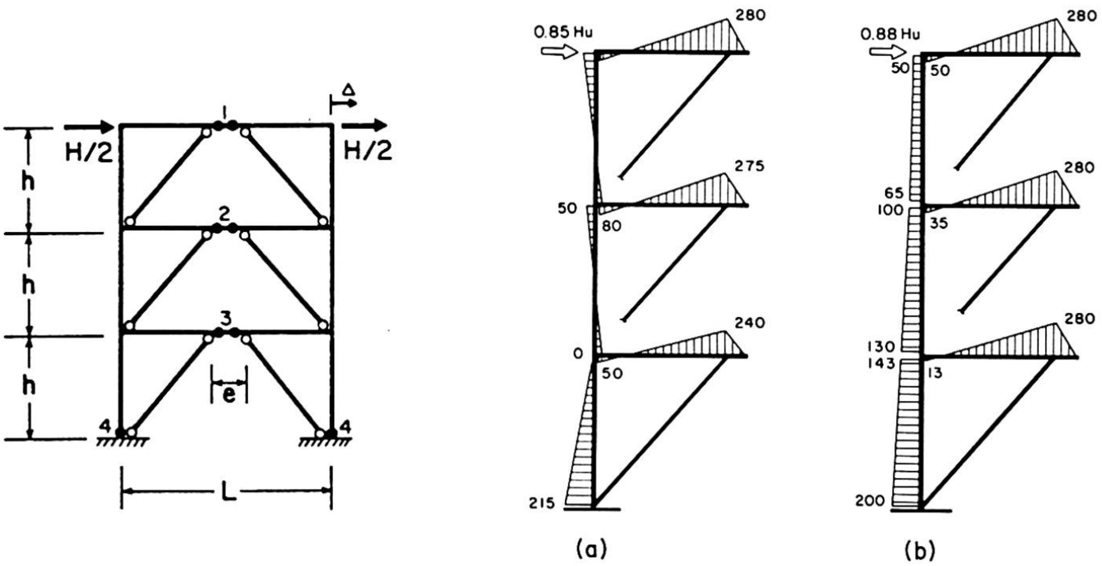


Fig. 3. Split K-Braced Frame. (a) Moments at First Yield. (b) Moments at Frame Ductility of 2 [8].

With the Canadians having already embraced plastic limit state as a basis for structural design, and with the US activity in the ACI and AISC in connection with the Load and Resistance Factor Design methods, the prospects for improving upon Eq. (1) seem good, and further developments in the plastic design methods appear imminent. The earlier emphasis on advocating plastic methods on the basis of economy seem ill-advised. On the other hand, such methods seem indispensible for a fuller comprehension of the problems in seismic design.

With a wider acceptance of plastic limit state as the basis in seismic design, one can foresee dynamic analyses becoming more sophisticated with new developments in the area of elasto-plastic response. At present, dynamic analyses are performed exclusively on the elastic basis.

The importance of a dynamic analysis can be illustrated by citing an example from Ref. 13. In this case, Paulay compares the distribution of moments in the columns as prescribed by the New Zealand code and what might happen during a severe earthquake (Fig. 4). The discrepancy is startling, calling for very different column reinforcement for the two cases. At 7.8 seconds, an elasto-plastic dynamic analysis shows no inflection points in the column along several stories, requiring a different pattern of reinforcement than that determined by the static analysis. The New Zealand code [17] makes special provisions for such a contingency.

Although this example is drawn from the design of a reinforced concrete building, the results are just as meaningful for steel columns. It is clear that under similar circumstances, the use of minimum column splices selected on the basis of static code analysis would be grossly inadequate.

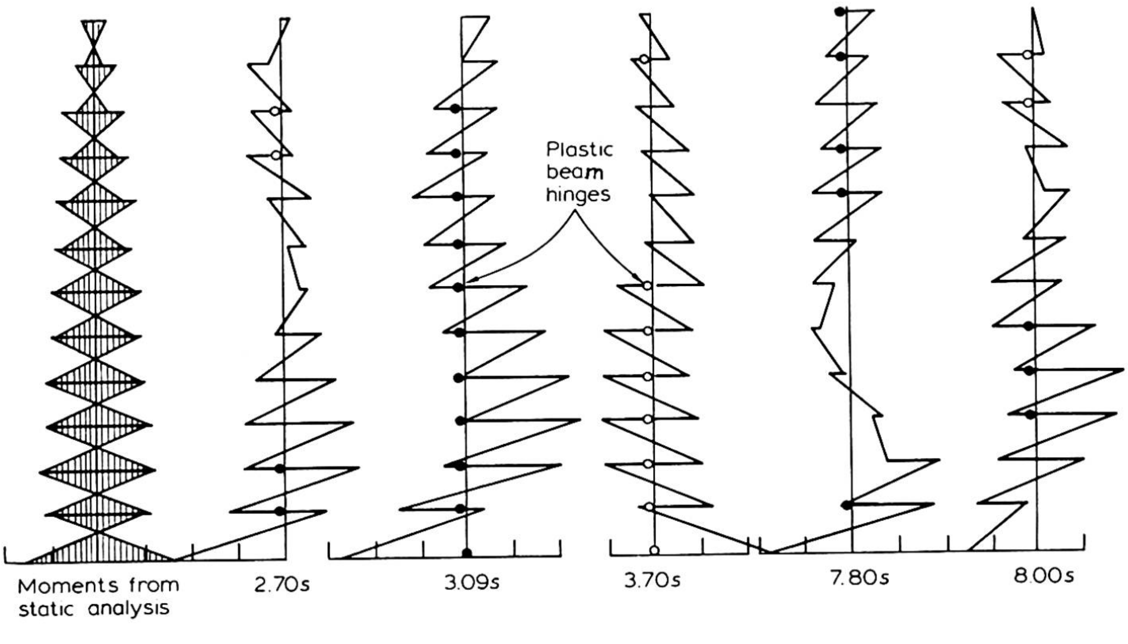


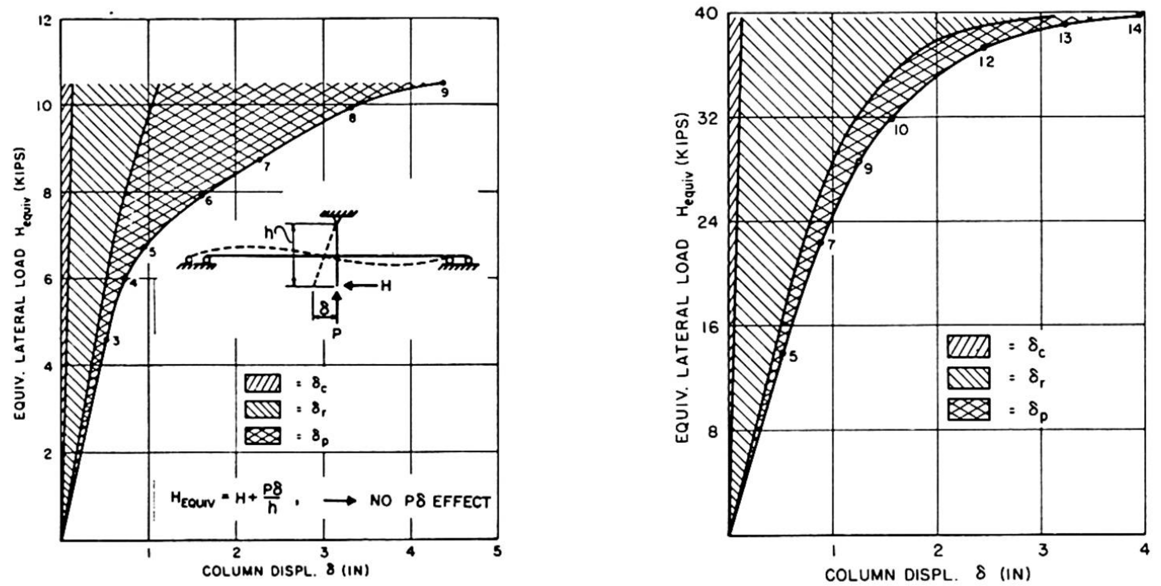
Fig. 4. A Comparison of Column Bending Moments During Instants of Large Earthquake Motion with Code-Specified Lateral Static Loading (after Paulay [13]).

5. ECCENTRICALLY BRACED STEEL FRAMES

As an illustration of the structural design process for seismic applications, the evolving novel scheme of bracing steel frames with diagonal braces having deliberate eccentricities at the joints will be discussed. First, the possible problems in the conventional steel framing for resisting lateral loads will be identified. Then the basic concept of eccentrically braced frames will be discussed, followed by an overview of the completed experimental studies of component behavior. Some feedback from full-size pseudo-dynamic experiments on a six-story steel building at Tsukuba, Japan [7], in which eccentric bracing was employed, will be presented.

In seismic design of structural steel framing systems for resisting lateral forces, either moment-resisting frames (MRFs) or diagonally braced frames are commonly employed. The MRFs are ductile, but tend to be too flexible, whereas the braced frames are stiff, but are not ductile. Therefore, both systems have an undesirable characteristic for seismic applications.

To optimize the behavior of MRFs, the panel zone, i.e., the column web between beam flanges, often requires reinforcement by means of doubler plates, and the beams may have to be made larger to control story drift. These aspects of the problem are illustrated in Fig. 5, where the contributions of the three main sources to story drift are identified for two beam-column subassemblage experiments [10]. These are the flexural deflection of the columns, δ_c, rotation of the beams, δ_r, and shear deformation of the panel zone, δ_p. For a thin panel zone (Fig. 5a), δ_p can contribute significantly to the story drift. This effect can be reduced by using larger columns or reinforcing the panel zones by doubler plates. If this problem is resolved, beam rotation δ_r becomes the principal cause of story drift (Fig. 5b), and it becomes necessary to use larger beams than required for strength. Both remedies are economically unattractive.



(a) (b)

Fig. 5. Horizontal Displacement Components of Column with (a) Thin Web and (b) Thick Web [10].

Diagonal bracing provides an effective means for reducing story drift, and is an excellent solution for wind bracing. However, for seismic applications, it has a major difficulty because the tensile braces are ineffective during repeated cyclic stretching, and compression braces lose their capacity under repeating and reversing post-buckling loadings. The behavior of a strut in post-buckling range is illustrated in Fig. 6, where an initially concentrically loaded strut is subjected to a number of severe load reversals. The large decrease in compressive strength of the strut during reloading is striking. This intrinsic lack of compressive capacity reliability of a strut under cyclic loading raises serious objections to concentrically braced frames (CBFs) for applications in regions of high seismicity.

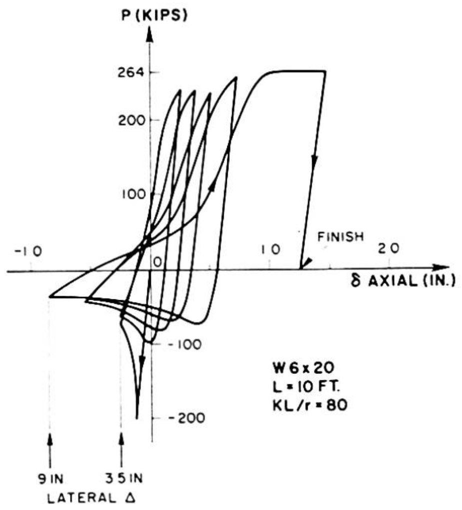
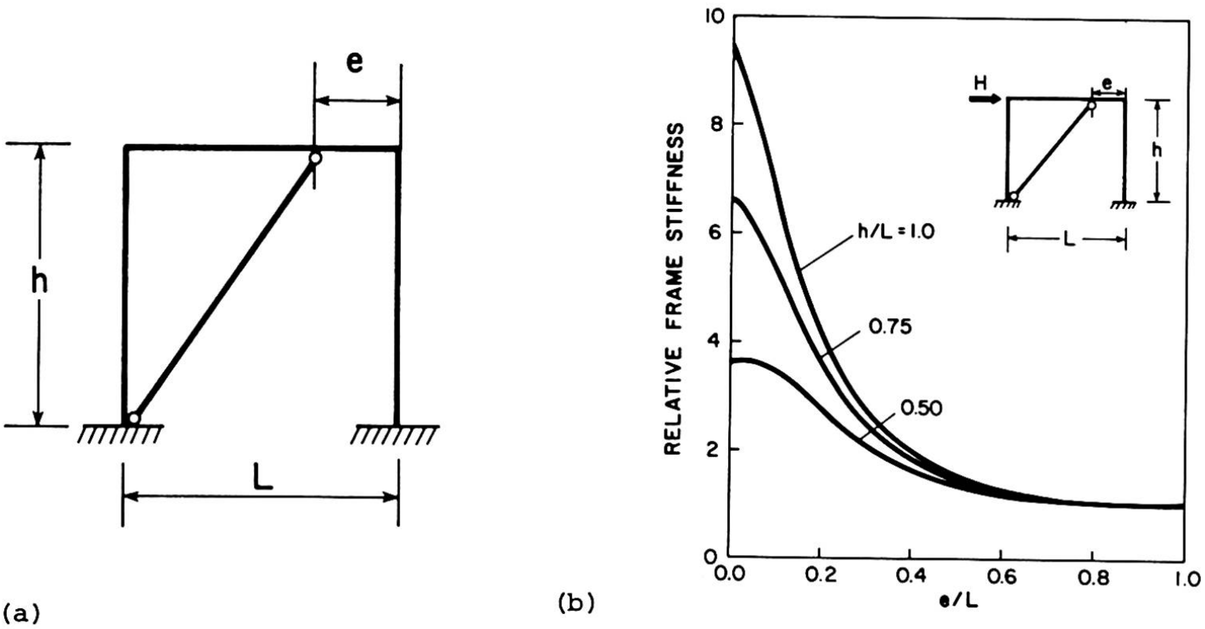


Fig. 6. Experimental Hysteretic Loops for Cyclically Loaded Strut [20].

A possible solution for steel framing for seismic design consists of a compromise between the two basic types of structural framing, i.e., between the moment-resisting and concentrically braced framing. This concept can be clarified by making reference to Fig. 7a [8], which shows the simplest eccentrically braced frame (EBF). When the brace eccentricity e is reduced to zero, one obtains the conventional CBF, whereas if e = L, one has an MRF. For all other values of e, the frame is an EBF. By making the diagonal member sufficiently strong so as not to buckle, but rather to cause yielding in the short link, the wanted behavior of an EBF is achieved. A parametric study of the elastic behavior of this simple frame is shown in Fig. 7b [8]. From this diagram, one



(a) (b)

Fig. 7. (a) Simple Eccentrically Braced Frame. (b) Variations of Stiffness for Different Aspect Ratios with Constant Member Sizes [8].

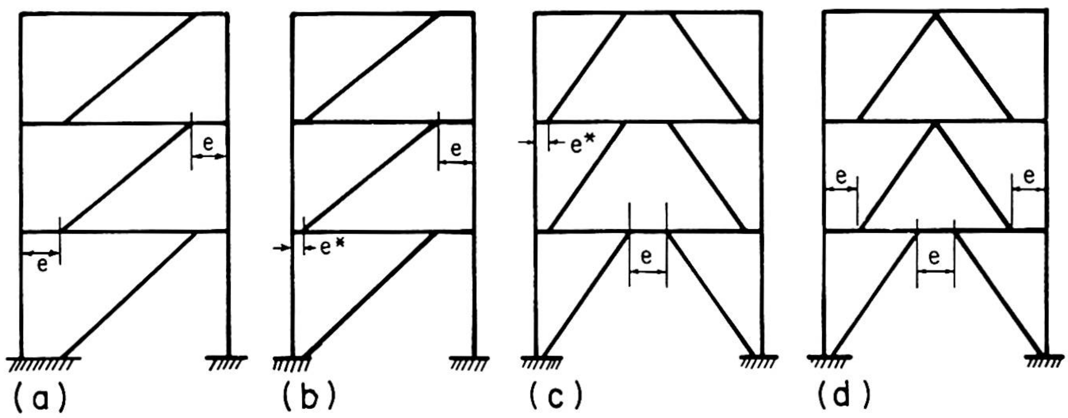
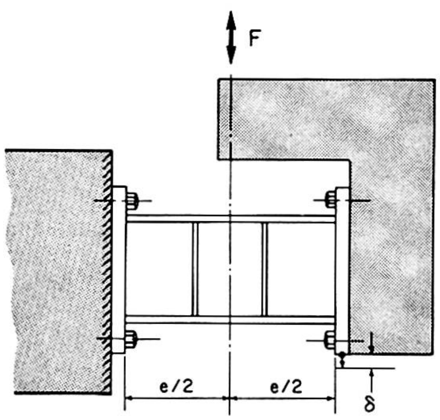


Fig. 8. Alternative Bracing Arrangements for Eccentrically Braced Frames [8].

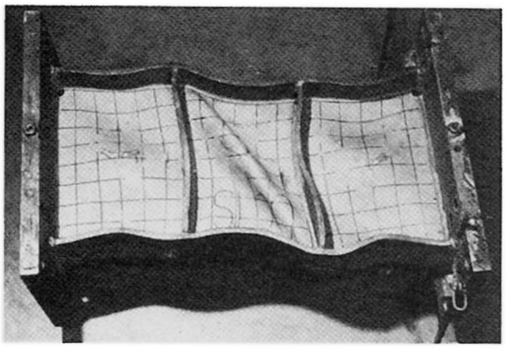
can note that a large frame stiffness can be achieved by an EBF. On the other hand, by making these links sufficiently long, they can sustain the required plastic deformations; thus, a ductile framing system is obtained. Extensive experimental and analytical research has shown that it is possible to achieve these objectives for a variety of EBFs. Some examples of such framing are shown in Fig. 8 [8].

The basic experiments for a typical link for a split-K framing (Fig. 8c) were performed using the idealization shown in Fig. 9a [8]. To retain frame elastic stiffness, the links should be made as short as possible (Fig. 7a), consistent with their ability to sustain severe plastic deformations. At cyclic overloads these short links must maintain their strength while webs yield plastically. As such behavior was not anticipated in the codes, appropriate rules for stiffening the web were developed [12]. An example of a correctly designed link at the end of a severe cyclic test is shown in Fig. 9b.

Additional experiments had to be performed on links occurring next to columns (Fig. 8a,b, and d). For such cases, in the elastic range of behavior, significantly larger moments develop next to the columns than at the brace end. The extent of moment equalization and web yielding was studied using the model



(a)



(b)

Fig. 9. (a) Schematic Diagram of Test Setup for Interior Link. (b) Well Stiffened Link at End of Severe Cyclic Test [8,12].

shown in Fig. 10b. By applying equal cyclic displacements at the two load points, the behavior of this isolated beam simulates the conditions that would develop in a frame in the inelastic range (Fig. 10a) [9]. For a better simulation of the link behavior in a building, experiments on steel beams with a composite floor were designed, and experimental work is in progress (Fig. 11) [15]. The adopted model is designed to simulate both the interior links (Fig. 11c) and those occurring next to the columns (Fig. 11b).

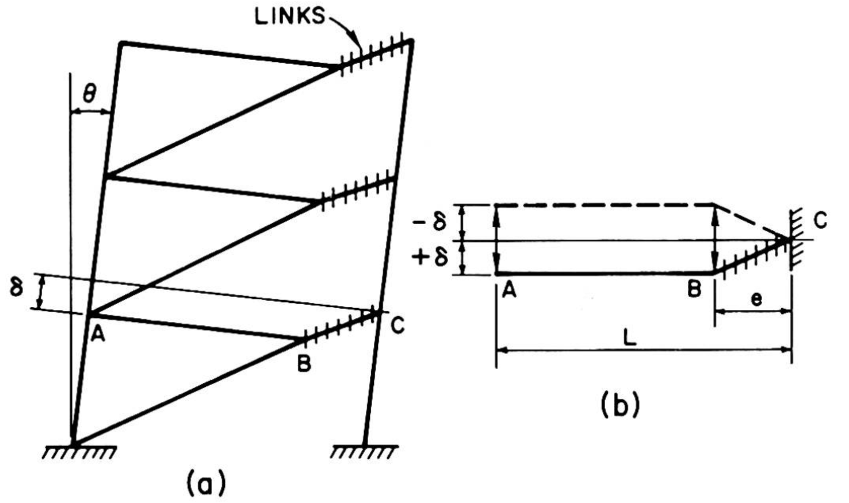


Fig. 10. (a) Collapse Mechanism of a Frame. (b) Schematic Diagram of Test Setup for Exterior Link.

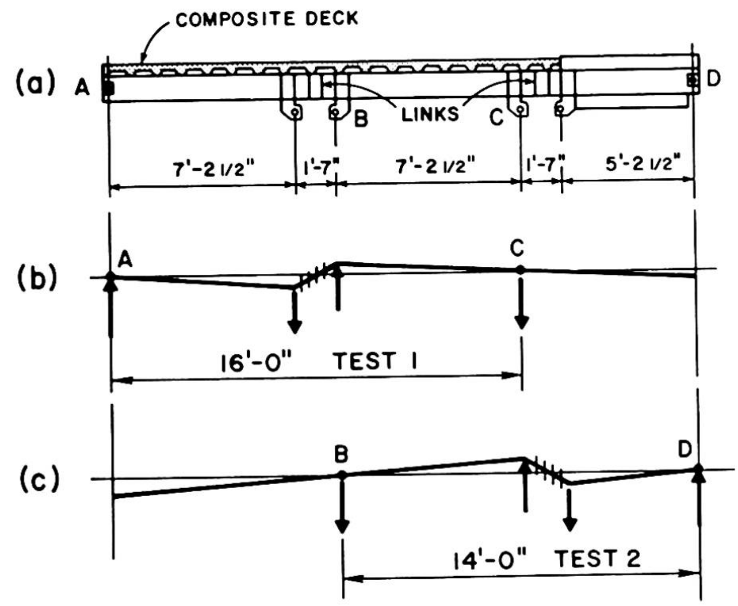


Fig. 11. (a) Composite Floor Beam with Links. (b) and (c) Schematic Diagrams of Test Setups for Interior and Exterior Links.

An analytical procedure for preliminary design of EBFs using plastic methods of analysis has been developed [9]. Experience using this procedure shows its versatility and great simplicity. Elastic analyses of plastically designed frames indicate excellent behavior of such frames. Relating these analyses to probabilistic evaluations of designs remains a challenging task.

6. SUMMARY

In practice, the structural design process with seismic considerations is reduced to simple terms by prescribing deterministic lateral static loads. Usually, an elastic analysis is performed for sizing the members. The deficiencies of this approach have been emphasized in this paper. As a first step, it is advocated to adopt a true concept of limit state design, which would usher plastic methods into the analysis and design process. Improvements in analytical solutions leading to better agreement with experimental results are sorely needed. Research on isolated members alone is no longer adequate. Experimentation must continue to be conducted at least at the level of subassemblages.

Hopefully, meaningful advances will be made in rapid dynamic inelastic analysis of structures. Correlations with pseudo-dynamic tests as well as with experiments on shaking tables are needed.

Engineers must become more aware of the size effect. To date, experiments have been performed on very small specimens. Particularly with steel, the concepts of fracture mechanics for low-cycle fatigue need to become generally appreciated. The problem of cyclic bond deterioration in reinforced concrete as well as a more precise knowledge of confinement effects need further attention. Until such time as these questions will be more accurately resolved, seismic design of structures will remain to some extent an art, and will continue to tax the ingenuity of the engineer.

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