THEME II

SEISMIC BEHAVIOUR OF STRUCTURAL CONCRETE LINEAR ELEMENTS (BEAMS, COLUMNS), AND THEIR CONNECTIONS

COMPORTEMENT SISMIQUE DES ELEMENTS STRUCTURAUX LINEAIRES (POUTRES, POTEAUX) ET DE LEURS JONCTIONS

Reporter
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SUMMARY

This state-of-the-art report summarizes knowledge of seismic behavior of linear reinforced concrete elements; points out studies which have already led to improved design and construction rules for seismic resistant structures; discusses problems without satisfactory solutions; and formulates recommendations for research and development. This report has been divided into eight main sections. The first section includes introductory remarks, objectives and scope. The second section reviews general aspects and problems involved in predicting seismic behavior of R/C linear elements and their connections. The third, fourth, and fifth sections review seismic behavior of beams, columns, and beam-column joints, respectively. In these sections only the behavior of elements cast with normal weight aggregate and ordinarily reinforced are discussed; lightweight aggregate elements are reviewed in section six. Section seven reviews seismic behavior of prestressed, and of precast elements and their connections. Finally a summary, conclusions, and recommendations for future research and development are presented.

RESUME

Ce rapport résume les connaissances du comportement séismique des éléments linéaires du béton armé. Il souligne les études qui ont conduit à une amélioration des méthodes de "design" et de construction des structures résistantes aux secousses séismiques. Ce rapport discute aussi des problèmes pas encore résolus, et formule des recommendations relatives aux besoins de recherche et de développement. Le rapport a été divisé en huit sections. La première section contient l'introduction, les objectifs et le cadre de l'étude. La deuxième passe en revue des aspects et des problèmes rencontrés dans la prédiction du comportement sèismique des éléments linéaires de béton armé et leurs connections. Les troisième, quatrième, et cinquième sections sont consacrées au comportement séismique des poutres, des colonnes, et des connections poutre-colonnes. Ces sections se limitent aux éléments de béton fabriqués avec des agrégats de poids normal, et armés de façon standard. La sixième section traite les éléments fabriqués avec des agrégats légers. La septième est consacrée au comportement séismique des éléments précontraints et préfabriqués, et leurs connections. Finallement, un résumé, des conclusions, et des recommandations pour les futures recherches et développements sont présentés.

1. INTRODUCTION

1.1 General

Significant advances have been made in the U.S. in the last two decades in understanding seismic behavior of concrete structures. This is particularly true for concrete structures of the moment-resisting frame type whose basic elements are beams and columns, their connections and supports, and interacting floor slabs. The stimulus for these advances were [1]; the First World Conference on Earthquake Engineering in 1956 [2]; the publications of 1959 SEAOC Blue Book [3]; the PCA Manual in 1961 [4]; and the damages caused by the following earthquakes: 1964 Alaska [5], 1967 Caracas, Venezuela [9], the 1968 Tokachioki, Japan, and 1971 San Fernando [6].

By reviewing the proceedings of the six world conferences on earthquake engineering [2, 7 - 11], a picture can be obtained of the advances made during the last two decades in predicting seismic behavior of concrete structures. These advances have been particularly noticeable in the last ten years, and have had some impact in earthquake resistant design of all kinds of concrete structures. However, much of present knowledge has not been practically applied, because usually there are several problems to overcome before research results can be introduced into codes and implemented in practice [1]. In spite of these problems there has been considerable progress in developing code requirements for earthquake resistant construction. This is reflected in new codes such as the 1976 Mexican Code [12, 13], the 1976 New Zealand Code [14], and the Tentative Provisions for the Development of Seismic Regulations for Buildings [15].

The work that has been done in the field of seismic behavior of structural concrete linear elements and their connections cannot be reviewed adequately or even summarized in one short paper. Therefore some restrictions will have to be placed in the objectives and scope of this report.

1.2 Objectives

The main objectives of this report are:

1) to summarize present knowledge of seismic behavior of linear reinforced concrete elements; pointing out those studies which have already led or may lead to improved design and construction rules for seismic resistant structures;

2) to discuss problems whose solutions are not yet satisfactory and to formulate recommendations for research and development needs.

1.3 Scope

To achieve the above objectives, first a summary was made of the data and results available from studies of the seismic behavior of linear reinforced concrete elements and their subassemblages, and complex structures made of such structural elements. The significance of these results was analyzed in light of the total problem of the design and construction of seismic resistant concrete structures. Ordinarily reinforced linear elements cast in place of normal weight concrete are considered first. Then the problems created by the use of lightweight concrete and the use of prestressing and precasting techniques are discussed.

This state-of-the-art report has been divided in to eight main sections.

1. Introduction
2. Review of General Aspects and Problems Involved in Predicting Seismic Behavior of Structural Concrete Linear Elements and their Connections
3. Seismic Behavior of Structural Concrete Beams
4. Seismic Behavior of Structural Concrete Columns
5. Seismic Behavior of Beam-Column Joints
6. Seismic Behavior of Structural Lightweight Concrete Linear Elements and their Connections
7. Seismic Behavior of Prestressed and Precast Linear Elements and their Connections
8. Summary, Conclusions and Recommendations for Future Research and Development.

A review of the state-of-the-art of experimental work on seismic behavior of linear elements up to 1972 has been presented by the author in Ref. 16. Therefore, in this report only accomplishments not reviewed in Ref. 16 and particularly those in studies published since 1972, will be presented. A review of studies carried out up to the beginning of 1977, as well as a discussion of the accomplishments and research and development needs at that year, has been presented in a workshop on "Earthquake-Resistant Reinforced Concrete Building Construction" (ERCBC) sponsored by the U.S. National Science Foundation [17]. Most of the present report is based on the papers, reports and discussions presented at the ERCBC workshop and those that have been published since then. Because of lack of time and space, it has not been possible to present an exhaustive review of the subject under consideration. Only general questions of the seismic behavior of linear elements are discussed herein. For a more detailed discussion the reader is referred to Ref. 17.

2. REVIEW OF GENERAL ASPECTS AND PROBLEMS INVOLVED IN PREDICTING SEISMIC BEHAVIOR OF STRUCTURAL CONCRETE LINEAR ELEMENTS AND THEIR CONNECTIONS

2.1 General Remarks

Seismic performance of reinforced concrete structures during moderate and severe earthquake ground motions has ranged from minor cracking to complete collapse. Due to these collapses, and the relatively lower strength and ductility per unit weight of ordinary (unconfined) reinforced concrete when compared with structural steel, it has been suggested that concrete structures are particularly vulnerable to earthquakes. However, as Bresler points out [18], many concrete structures have withstood severe earthquakes without significant damage, suggesting that there is nothing inherent in concrete structures which makes them particularly vulnerable to earthquakes. Regardless of the material used, properly designed structures will perform well, although reinforced concrete may be less "forgiving" or less "tolerant" of improper construction (workmanship) and maintenance. Due to this sensitivity, to have reinforced concrete structures that will behave satisfactorily under severe earthquake ground motions it is necessary that designers pay special attention to all the factors that can affect seismic structural performance. It is not enough to design structures in accordance with the requirements of the latest seismic codes. Performance of a structure depends on its state when the earthquake strikes, which may well be significantly different from the state the designer thought would exist at that time. Thus, modifications, maintenance, and repair of structures during their lives must be considered in addition to the general aspects involved in their construction.

The general philosophy of earthquake-resistant design for buildings other than essential facilities has been well established and proposes to: prevent nonstructural damage in frequent minor earthquake ground shakings; prevent structural damage and minimize non-structural damage in occasional moderate earthquake shakings; and avoid collapse or serious damage in rare major ground shakings. This philosophy is in complete accord with the concept of comprehensive design. However, current design methodologies fall short of realizing the objectives of this general philosophy [19].

In a comprehensive design approach it should be recognized that building damage may result from different seismic effects: (1) ground failures due to fault ruptures or to the effects of seismic waves; (2) vibrations transmitted from the ground to the structure; (3) seismic sea waves (tsunamis) and tsunami-like disturbances and seiches in lakes; and (4) other consequential phenomena such as fires, and floods caused by dam failures and landslides.

The seismic effect that usually concerns the structural engineer, and is accounted for by seismic-resistant design provisions of building codes, is the response or vibration of a building to the ground shaking that might occur at its foundation. Although damage due to other effects may exceed that due to vibration of the building, this paper considers only the effects of ground shaking at the foundation of concrete structures.

2.2 Construction and Maintenance Aspects

As pointed out earlier, a building's response, and the damage it sustains during any kind of excitation depends on how the building was actually constructed, not on how the designer thought it would behave. Furthermore, design and construction are intimately related: the achievement of good workmanship depends, to a large degree, on the simplicity of the detailing of members, connections, and supports. This is especially true for reinforced concrete structures. Although it is possible (on paper and even in laboratory specimens) to detail reinforcement in such a way that seismic behavior is considerably improved, in the field such design details may be too elaborate to be economically feasible. A design can only be effective if it can be constructed.

Field inspection has revealed that a large percentage of damage and failure has been due to poor quality control of structural materials and/or poor workmanship, problems which could have been corrected if the building had been carefully inspected during construction. In many other cases, damage, even failure, may be attributed to improper maintenance of buildings during their service life [20].

Analyses of mill tests of reinforcing steel bars, field control tests of concrete cylinders, and mechanical materials studies of specimens removed from the structures, show considerable variation in mechanical characteristics [20]. In view of this variability, present seismic code provisions which specify only minimum and maximum material strengths, and recommend that the design and capacity of members be based on these code specified strengths alone, are unreliable and can lead to unsafe designs. This is especially true in designing connections and in designing for shear for reinforced concrete structures.

2.3 General Features of Seismic-Resistant Design

Efficient seismic-resistant construction necessitates careful attention to the total seismic design, construction, and maintenance process. The phases of this process include: evaluating the seismic threat, selecting the structural layout and predicting the mechanical behavior of the whole soil-building system; proportioning and detailing the structural components, with their connections and supports; analyzing the reliability of the design obtained; and constructing and maintaining the building during its service life.

The main design aspects that should be considered are summarized in the flow diagram show in Fig. 1. This diagram shows the intimate relationships of the different aspects and steps in the design process. The inelastic response of a structure and therefore of its components, is extremely sensitive to the dynamic characteristics of ground motion to which it is subjected, as well to mechanical characteristics of its structural and nonstructural components. Studies of the response of concrete structures to severe earthquakes show that the performance of any specific critical region of an element is not only very sensitive to the gound motion and the mechanical characteristics of the structural materials but even to the final main and secondary reinforcement. This sensitivity must be recognized in order to properly analyze the significance of the results to be presented later.

In most designs the loading that will occur during the life of a structure is uncertain and can only be described in probabilistic terms. This is especially true for seismic loadings as the uncertainties are unusually large. Clearly, it would be rational to take a probabilistic approach to seismic design [32]. To do this it will first be necessary to collect sufficient statistical data.

In summary, the seismic behavior of any element depends on the interaction of the ground motion and the structure to which this element belongs [19 - 22]. The seismic behavior of any structural element cannot be predicted solely on the basis of its mechanical characteristics: the pattern (shape function) of the excitations (forces, deformations, changes in environment, etc.) to which it is subjected must also be defined. Establishing the probable critical excitation involves significant uncertainties and can only be described in probalistic terms.

In judging the reliability of results from experiments carried out on reinforced concrete, and particularly in assessing the implications of these results for design and construction of real seismic resistant structures, it is necessary to consider the nature and degree of uncertainties in the actual mechanical characteristics of the structural materials (concrete and reinforcing steel) that are used. To define these uncertainties there is a need for extensive collection of field and laboratory experimental data, for statistical study of these data [23] and the variations obtained from them. Studies may then be carried out on the probability of failure of reinforced concrete elements [24]. The uncertainties of seismic loading, combined with uncertainties of actual mechanical characteristics of structural materials, and of mathematical modeling of structural element behavior, make predicting seismic behavior of even the simplest structural elements (such as the linear elements under consideration in this report) extremely complex.

2.4 Importance of Loading History

As pointed out by Park [25], "in the past a variety of loading sequences and acceptance criteria has been used by various research laboratories throughout the world, making the comparison of results difficult and resulting in different conclusions from the obtained results". In Refs. 16, 22 and 26, the author has discussed and illustrated the effects of loading history on different types of reinforced concrete elements and subassemblages. Figure 2 illustrates the effect of different loading histories on the behavior of reinforced concrete columns [27,28]. Similar effects on a beam-column subassemblage of a lightweight reinforced concrete ductile moment-resisting frame are shown in Fig. 3 [29].

In Ref. 28 Jirsa discusses the effect of various loading systems. The importance of this problem was brought out in the workshop on Earthquake-Resistant Reinforced Concrete Building Construction, conducted in Berkeley in 1977 [17]. Several working groups made strong recommendations regarding standard loading histories and acceptable criteria for judging seismic behavior of strucural elements (see Volume 1 of Ref. 17).

2.5 Importance of Proper Structural Layout

As summarized in the flow diagram of Fig. 1, in this step the designer must select the structural system, the structural material, and the type of non-structural components. Proper solutions of the problems encountered in this step require close collaboration between the architect, structural engineer, and manufacturers of materials. Sophistication in the selection of the structural system, structural material, and nonstructural components is of much greater importance than sophistication in analysis. Regardless of how sophisticated a method of analysis an engineer uses he cannot make an ill conceived structural system behave satisfactorily in a severe earthquake. The inertial forces depend upon the mass, damping, and the structural characteristics themselves (stiffness, yielding strength, maximum strength, and energy absorption and energy dissipation capacities). Therefore, decisions made regarding the choice of layout for the structure and the choice of structural and nonstructural material must play a significant role in the seismic performance of the structure during its lifetime. One of the best policies in earthquake resistant design is to avoid problems whose solutions are unreliable or not known. Therefore designers need to understand how design decisions may create serious seismic effects. A discussion of the selection of a proper structural system is offered in Refs. 20 and 30.

Structural material should have high energy absorption and energy dissipation capacities per unit weight. To achieve these high capacities the material should possess: (1) High strength (tension and compression) per unit weight; (2) High stiffness per unit weight; (3) High internal damping per unit weight; (4) High toughness per unit weight; (5) High resistance to low-cycle fatigue; and (6) Stable hysteretic behavior under repeated strain reversals. Furthermore, the structural material should be homogeneous, and easily adaptable and conducive to forming full-strength connections having the same characteristics as the material itself [20,30]. In selecting the best structural material for earthquake resistant construction, a simple plot of the ratio of stress per unit weight versus strain for the different available structural materials can be of use (Fig. 4). Plain normal weight concrete is not a desirable structural material for this type of construction because its weakness in tension requires that it be reinforced. The usually small ductility of ordinary reinforced concrete dictates the use of confined reinforced concrete.

The relatively low value of the strength per unit weight of normal weight concrete suggests the desirability of using lightweight concrete. The advantage of using confined lightweight aggregate concrete can be seen from the results in Fig. 4.

Since reinforced concrete is a composite of reinforcing steel bars and concrete (which is itself a composite material), there are many different types of reinforced concrete material in use today. Possible combinations depend on the different types of aggregate concrete (normal or lightweight) and reinforcing steel (prestressed or non-prestressed) which are used, and if the concrete is cast on site or precast. Precast, partially prestressed lightweight aggregate concrete is the combination most likely to be of greatest use in the future. The technology of lightweight aggregate and the problems of connections of the prefabricated elements, however, have not yet been resolved properly, and at present the most suitable reinforced concrete material for earthquake resistant construction is ordinary reinforced normal weight concrete [20].

2.6 Importance of Studying Behavior of Basic Structural Elements

The importance of studying the behavior of basic structural elements of a complete structure has been considered in Refs. 16, 26, 31 and 32.

It may appear ideal to test real buildings under the actual loading conditions to which they may be subjected during their service life. Such tests are not usually feasible due to economic considerations. Even if it would be economically feasible to test a replica of a given building, very little would be gained as far as improving our general understanding of seismic behavior of the different types of concrete buildings that are in use. There are at least two drawbacks to testing real buildings beyond the economic problem. The first is that the overall seismic response of an actual bare structural system is usually altered or obscured by the participation of nonstructural elements; and also the performance of individual structural elements can be modified by interaction with nonstructural components. This makes it difficult to extrapolate the information gathered to different types of structural systems or even to the same type with different kinds of arrangements of nonstructural elements. Therefore, it becomes desirable to obtain a detailed understanding of the behavior of the basic elements of a structure under loading or deformation similar to that expected from severe seismic ground motion, and how this behavior can be altered by the presence of nonstructural components.

The second drawback to testing real buildings is that information gathered by testing a complete building or structure is difficult to evaluate and usually insufficient to forecast the structure's behavior. Prediction necessitates the development of a theoretical model that may best be developed from observed behavior. A logical way to develop such a theoretical model is to start from an understanding of, and work toward a realistic idealization of, the behavior of simple elements [32].

3. SEISMIC BEHAVIOR OF STRUCTURAL CONCRETE BEAMS

3.1 General

In Ref. 16 the author discussed studies up to 1972 regarding the behavior of structural concrete linear elements and subassemblages. In this reference, rather than discussing results obtained from linear elements (beams and columns) the author has chosen to discuss the behavior of critical regions of these elements, classifying them as: (1) Flexural Critical Regions; (2) Flexural Critical Regions with High Shear; and (3) Flexural Critical Regions with High Axial and Shear Forces. While the first two types of critical regions are developed in beams they can also be associated with columns that are subjected to low axial forces. The third type of critical region usually develops only in columns.

Since 1972 tremendous amounts of research into seismic resistance of structural concrete elements, particularly beams, has been conducted in many countries [17]. In spite of the advances in knowledge due to this recent research, there is still a lack of agreement about some basic questions of seismic behavior of structural concrete elements. Some of these basic questions are described below.

1. What are the best indices for measuring seismic behavior of structural concrete elements?

Moment vs. curvature of the critical sections

Moment vs. rotation of the critical regions

Shear vs. rotation of the critical region

Shear vs. shear distortion of the critical region

Load-displacement relationship of the whole element.

2. What are the critical loading conditions that control the desired seismic behavior?

Monotonically increasing

Cyclic with no force reversal

Cyclic with force reversal, but not deformation reversals

Cyclic with partial deformation reversals

Cyclic with full deformation reversals

3. Which is the critical limit state, i.e. the one that will control design? Is it the serviceability, the damageability, or the collapse limit state?

4. What is the acceptance criterion for strength capacity and deformation, or energy absorption and energy dissipation capacities (ductility)?

What is the maximum deformation (ductility) that can be tolerated or demanded? How many cycles of full load or deformation reversals at different ductility levels are required?

These questions are a result of uncertainties regarding: (1) dynamic characteristics of future earthquake ground motions; and (2) structural response. These uncertainties make it impossible to accurately : (1) simulate the effects of real ground motions on the members of real structures (i.e the loading or deformation histories to which actual structural members are subjected; and (2) to predict the level of inelastic deformation (ductility) demand which will occur.

No definite or proper answers to the above questions have been offered. A variety of parameters of loading sequence levels; of intensity of deformations; and of acceptance criteria have been and are still used today by different researchers, making comparison of research results very difficult.

In studying the seismic behavior of beams, or more precisely, the hysteretic behavior of flexural critical regions of linear elements which can be subjected to severe seismic excitations, the author has used the moment vs. rotation of the critical regions, and the limit states and loading sequences shown in Fig. 5, particularly Fig. 5(b). Acceptance criterion has varied according to the structural system, the function of the structure, and the seismic risk at the structural site.

Two loading criteria which have been used in New Zealand laboratories in pseudo-static load tests [25] are shown in Figs. 6(b) and 6(c). The ductility factor is calculated using the first yield displacement for the first inelastic load run defined as in Fig. 6(a). The simple loading criterion shown in Fig. 6(b) involves initial loading runs in the elastic range to establish the initial elastic stiffness and then four loading runs in each direction to displacement ductility factors of 4. This criterion follows that recommended in the New Zealand Loading Code [14]. The more complex loading criterion shown in Fig. 6(c) involves more elastic loading runs to observe stiffness changes between the cycles of imposed inelastic deformations, and cycles of imposed inelastic deformation with gradually increasing displacement ductility factor. A simple acceptance criterion is that the seismic load carrying capacity should not reduce by more than 20% during the test [14].

Agreement on the above issues among researchers and designers would enable research results to be compared on a consistent basis. This agreement may only be achieved when sufficient and appropriate data on the above factors becomes available. This will necessitate an extensive analytical and experimental program. Such a program requires nonlinear dynamic analysis of the whole structure in order to determine the realistic range of expected demands on the members (e.g. loading conditions, ductility levels, pattern, and number of reversals). Analysis of results from some of the nonlinear dynamic studies carried out at Berkeley on moment-resisting frame [33] and coupled wall structural systems [34] have shown the sensitivity of plastic rotation demands with respect to the dynamic characteristics of the ground motion and mechanical characteristics of the structure (Figs.7 and 8). When a frame beam (a beam belonging to a ductile moment-resisting frame) is designed using present code requirements, the number of yielding excursions required and the number of rotation reversals that the critical region of the beam undergoes is usually smaller than four even under the most severe earthquake. However, the number of reversals increases considerably for critical regions of beams acting as the coupling of a coupled shear-wall structural system (Fig. 8).

Following is a review of studies since 1972 that are connected with the seismic behavior of beams in R/C moment resisting frames. Studies concerned primarily with beams used as coupling elements of coupled shear wall systems will be discussed at the end of this section.

3.2 Seismic Behavior of Ordinary R/C Beams in Moment-Resisting Frames

This section reviews studies of seismic behavior of ordinary R/C beams in moment-resisting-space frames, that have been cast in situ using normal weight aggregate concrete. Current knowledge on the behavior of lightweight aggregate concrete beams will be discussed in a later section. As in Ref. 16, the author found it desirable to distinguish behavior of beams whose critical regions are under practically pure flexure from those in which high shear exists.

3.2.1 Beams with Critical Regions whose Inelastic Behavior is Controlled by Flexure. In this paper, cases where the nominal unit shear stress in the critical regions is small [say $\leq 3\sqrt{f'_c}$ (psi) ($0.25\sqrt{f'_c}$ (MPa))] are distinguished from those in which this shear stress is high [16]. In the first case, one can neglect the effect of high shear on the hysteretic behavior of beam critical regions designed and detailed according to present seismic codes [35,36], even for the most severe seismic ground motion that can be expected during the life of the structure. The effect of high shear is usually detrimental to the energy dissipation capacity of critical beam regions. Therefore, it is important that designers working with the architect use a low percentage of reinforcement or select a structural layout (e.g., relatively long beams), which will experience low shear.

Numerous studies have been conducted on the behavior of R/C critical regions whose inelastic behavior is controlled by flexure[16,25,28,37-42,47] Although tests and analytical studies carried out since 1972 did not result in any significant new knowledge beyond that reported in Ref. 16, they did clarify some of the observations made in this reference and they also resulted in major advances in the design and detailing of R/C critical regions, specifically in improving the energy dissipation capacity of these regions. Some of these advances are discussed below under different parameters that affect the design of beams in frames. These parameters include; flexural strength capacity; deformation capacity; energy dissipation capacity; and prediction of stiffness necessary for predicting the dynamic characteristics of the frames.

3.2.1.1 Flexural Strength Capacity. In general, flexural strength can be predicted accurately if the mechanical characteristics of the reinforcement under uniaxial tension and compression, and of the concrete (confined and unconfined) are known [40,43-45]. However, there are still some uncertainties in evaluating the strength capacity of beam critical regions for slender beams in actual structures subjected to real earthquakes. Some of the most important uncertainties are: (1) the effect of the strain rate, particularly in cases where the maximum strength is reached at or just beyond the yielding of the reinforcing bars [16,46]; (2) the effective width of the floor slabs participating in the development of flexural capacity, especially the amount of slab reinforcement contributing to the total tension force.

As pointed out by Paulay [47], "the effectiveness of slab steel placed further away from the web of a beam or the face of a column is likely to depend on the torsional resistance of the beams framing into a column at right angles to the beam, the strength of which is being considered". No definite provisions for the effectiveness of slab steel exists in present codes, although different proposals were made in the draft recommendations of the New Zealand Code [14].

3.2.1.2 Deformation and Energy Dissipation Capacities. Deformation and energy dissipation capacities are controlled by the buckling of the main reinforcing bars, which is difficult to predict. Therefore, it is more difficult to predict these capacities than the flexural strength [22]. To avoid early failure and to assure sufficient deformation (ductility) and energy dissipation capacities most of the codes have introduced more stringent recommendations regarding proportioning and detailing of flexural critical regions. Some of the most important modifications are related to the following factors:

(i) Amount of Longitudinal Reinforcement Analytical and experimental results indicate that to attain large curvature ductility (say higher than 10, which is likely to be developed in the case of severe earthquakes) it would be preferable to use lower tension steel contents than are allowed by present ACI 318-77 [35] or 1976 UBC [36] provisions. (The ACI requires that the reinforcement ratio, ρ, shall not exceed 0.50 of the reinforcement ratio, ρ_b. The UBC requires that ρ shall not exceed 0.75 of ρ_b, or 0.025). Park [25] has reported that in New Zealand, if the compression steel ratio, ρ', is 0.5 of the tension steel ratio ρ, it is recommended that $\rho < 0.016$ when f'_c = 3,6000 psi (25 MPa) and $\rho < 0.022$ when f'_c = 5,800 psi (40 MPa), with linear interpolation between for other concrete strengths. When $\rho'/\rho > 0.5$, higher ρ values can be used. These limits are given by formulae based on analytical results [48,49].

(ii) Positive Moment Capacity at Column Connections According to ACI 318-77 [35] and the 1976 UBC [36] the positive moment strength of flexural members at column connections shall not be less than 50% of the negative moment strength. Analyses of experimental results [40] have indicated that to improve the energy absorption capacity of frames it is desirable to require that the ρ'/ρ shall not be less than 0.75. The advantage of designing beams in frames with this larger positive moment capacity has also been pointed out by Paulay [47] and shown by analytical studies conducted by Anderson [50] and by studies of ductility demand in actual design of framed structures [51].

(iii) Moment Redistribution in Beams of Frames Paulay [47] has pointed out that in the efficient design of reinforced concrete continuous framed beams there are three aims that the designer should attempt to achieve:

(a) Reduce the absolute maximum moment, usually in the negative moment region, and compensate for this by increasing the moment in the non-critical (usually positive) moment regions.

(b) Equalize the critical moment demands in beams at either side of an interior column

(c) Fully utilize the potential positive moment capacity of beam sections at the column faces.

In Ref. 47 Paulay discussed how to carry out efficient moment redistribution and concluded that this redistribution not only leads to a more economical beam steel arrangement but minimizes moment input in the columns.

Bertero and Zagajeski [52] have discussed the importance of proper selection of moment redistribution in applying a recently developed computer-aided optimum design procedure [53,54] to the seismic resistant design of R/C multistory frames. According to results obtained using different moment redistributions, it appears desirable to base the design on a constant amount of top and bottom reinforcement throughout the span. The design based on this approach requires an increase in total steel volume of about 16% without any increase in concrete volume. However, nonlinear dynamic analyses of the responses of the designed frames to different types of ground motions show that the design, based on having same amount of reinforcement along the whole span, performs better than a design in which the beam reinforcement is curtailed according to a moment envelope in which full moment redistribution is assumed. Although the required steel volume is higher, there are some additional costs in designs where the longitudinal reinforcement is curtailed. One of these is the time and therefore cost for properly detailing the reinforcement. Another is the cost of fabricating the steel cage in the field. Furthermore, the probability of human error is greater in detailing and fabricating the reinforcement cutoff than in placing continuous bars. A cut bar introduces a discontinuity in stress, and the section in which this cut takes place can become critical during seismic response. Taking all these factors into consideration, it is believed best (at least in U.S.) to use a constant amount of reinforcement along the length of the beam.

(iv) Transverse Reinforcement It is generally accepted that to obtain large deformation and energy dissipation capacities it is necessary to provide the beam critical regions (potential plastic hinge region), with properly designed, detailed, and fabricated transverse reinforcement. The larger the demand in deformation and energy dissipation capacities, the more stringent the requirements regarding transverse reinforcement should be. This type of reinforcement is required to provide:

(a) Confinement of Concrete: The larger the degree of confinement, the higher the compressive strength and particularly the deformation capacity of the confined concrete. Confinement also improves the bond characteristics of concrete.

(b) Lateral Restraint of Main Longitudinal Reinforcing Bars: The closer the spacing of the transverse reinforcement, the larger the buckling stress resistance of the main bar.

(c) Shear Resistance: The larger the amount of reinforcement, and the closer the spacing, the larger the increase in shear resistance.

A brief discussion follows of recent advances in each of these roles of transverse reinforcement.

(a) Confinement of Concrete: Although the beneficial effects of transverse reinforcement on deformation capacity (ductility) are well known and generally accepted, its influence on strength enhancement is not so well accepted. There is still controversy about the actual increase in ductility and particularly in strength. Reasons for some of the existing controversies are discussed by Bertero and Vallenas in Ref. 55.

In seismic-resistant R/C structures, the designer is interested in the confinement of longitudinally reinforced, not just plain, concrete. Therefore, there has recently been a series of studies on confinement of longitudinally reinforced elements [56-58]. These studies have improved understanding of the effect of different types and arrangements of lateral reinforcement on reinforced concrete. From these investigations, new constitutive laws for confined concrete have been suggested (see Fig. 9). In spite of these advances, there still is not sufficient data about hysteretic behavior of confined reinforced concrete under the state of stress and strain expected in flexural critical regions [55]. Present American seismic code requirements for beam confinement do not appear to be adequate when large ductility is demanded, although the 1976 UBC code requirements for the spacing of transverse steel (i.e. d/4, 8 bar diameters or 12 in.) are somewhat more stringent than the ACI-318-77 (i.e. d/4, 16 bar diameters or 12 in.).

(b) Lateral Restraint of Main Longitudinal Bars: For flexural critical regions designed and detailing according to present seismic code requirements, the strength and deformation capacities under seismic excitation are usually controlled by buckling of the main reinforcement [16]. The main factors controlling this buckling are: (a) bar size and its mechanical characteristics; (b) concrete cover; (c) the spacing, size, detailing, and fabrication of hoops (ties); (d) strain history of the steel bar; (e) length of the critical region (moment gradient). Based on experimental and analytical studies the author suggested in Ref. 59 that to prevent buckling of the main bar before it is strained to the beginning of strain hardening, the maximum spacing, s, should not exceed 4D to 8D for grade 40 steel and 3.5D to 7D for grade 60, where D is the diameter of the bar. As can be inferred from Fig.10, the higher limit values are obtained assuming that the ties offer perfect restraint - i.e., they do not allow any lateral movement of the main bar, which is difficult to achieve in the field. Inspection during fabrication of steel cages, even in the laboratory, reveals that there is always a gap between the corner of a tie and the main bar. Furthermore the legs of the ties will always deform or even move, particularly when the critical region is subjected to large inelastic deformation.

If large ductility is demanded (requiring a strain higher than that at the initiation of strain hardening), s should be even smaller than that indicated above (see Fig. 10). This is because the E_t at the required large inelastic strain (beyond the initiation of strain hardening) will be smaller than the $E_{ST.H}$ of a bar loaded monotonically in uniaxial compression.

Similar observations have been offered by other investigators [25,28, 43,60,61]. A detailed discussion of experimental observations regarding buckling is offered in Ref. 43. In the experimental results reported in this Ref. in which buckling was observed, the shear stress in the critical regions was higher than 3 $f'_c\sqrt{(psi)}$ $(0.25\sqrt{f'_c\ (MPa)})$.

Because of the results of beam tests [38-41,46,62], it was suggested that each bar should be supported laterally by a corner of a tie. Similar recommendations have been made by Park [25]. Park has also recommended that to prevent buckling in plastic hinge zones, the spacing of stirrup ties surrounding compression steel bars should not exceed six times the diameter of the bar. It is also recommended that stirrup ties spaced at 4 in. (100 mm) should have a force at yielding at least one-sixteenth of the yield force of the longitudinal bar which they laterally restrain. (Note that four inch spacing is not required by present U.S. seismic codes.)

The above requirements have been derived for and applied to beam critical regions of structures that can be subjected to very severe ground motions and in which these regions are the main source of energy dissipation i.e., ductile moment-resisting frames, and therefore they are too stringent for other cases. For this reason, the author believes that the formulation of seismic code requirements by just one set of empirical rules, although desirable for practical applications, usually leads to the use of provisions which are too stringent for most of the structures. Attempts should be made to formulate seismic regulations as a function of the demanded or desired deformation and energy dissipation capacities. Although this may not be possible due to insufficient data and unreliable methods of estimating actual demands, work in this direction should be started.

(c) Shear Resistance: Advances in design requirements for this role of transverse reinforcement will be examined later in a discussion of flexural critical regions under high shear.

3.2.1.3 Prediction of Stiffness. To estimate the demands of strength, deformation, and energy dissipation capacities during an earthquake, it is necessary to predict the dynamic characteristics of the structure at the time when the earthquake shaking takes place. In case of R/C ductile moment-resisting frames, the period of the frame is usually controlled by the stiffness of the beams rather than that of the columns. Thus the initial stiffness of the beams at the moment an earthquake occurs must be estimated as accurately as possible. In as much as the time of occurrence of future earthquakes and the history of excitation to which the structure has been subjected are uncertain it is best to consider a range of probable values of stiffness in analysis. It is even possible that certain beam regions in a structure may have been critically stressed due to service conditions alone (e.g. high service loads, thermal or settlement deformations, environmental conditions) leaving these beams in a state of reduced stiffness. Unfortunately this is an area in which very little progress has been made.

Several investigators have developed sophisticated methods to predict moment vs. average curvature at critical regions and have obtained good agreement with measured values [40,44,45]. This is illustrated in Fig. 11(a) for the case of monotonically increasing moments, up to large displacement and then unloading and in Figs. 11(b) and 11(c) for the case of cyclic bending reversals. However no satisfactory rules have yet been offered to use in practice for predicting initial stiffness. The lack of such rules and the consequent implications for practical seismic design have been discussed by Strand [51]. Strand points out that neither the degree of cracking nor the effective width of the flanges due to floor slab contribution are well defined in either the code or in available literature.

A method for computing lateral stiffness of beams under seismic excitations has been presented by Orudgev and associates in Ref. 63 and appears to give good results. However, again no clear rules are given for estimation of initial beam stiffness.

3.2.2 Beams with Flexural Critical Regions under High Shear. When nominal unit shear stress at the critical region of a framed beam exceeds $3\sqrt{f'_c}$ (psi) ($0.25\sqrt{f'_c}$ (MPa)), and the beam has been designed and detailed according to present U.S. seismic codes, the critical region is capable of developing maximum flexural strength and flexural deformation capacity under monotonically increasing loads. However, this kind of critical region, under repeated moment reversals and particularly under full rotation reversals, will undergo a degradation in stiffness, and energy absorption and dissipation capacities considerably larger than the degradation of a similar critical region with very low shear stresses. Degradation in strength starts to occur as the number of similar loading cycles inducing reversal of rotation increases. Although such regions are capable of developing flexural yielding strength, an early shear failure mechanism (sliding shear) starts to develop after one cycle of full bending reversals beyond the yielding strength level. This behavior was discussed in Ref. 16 in 1972 but at that time it was pointed out that the data available was scarce. Sources given at that time include Refs. 64-67. Since 1972 significant advances have been achieved thanks to the work of many researchers. Some of the results obtained have been published and discussed in Refs. 25, 28, 37-43, 47, 62, 68-72.

A detailed discussion of the effects of high shear in flexural critical regions is given in Ref. 40. Figures 12 and 13 illustrate the results obtained from two beams, R-5 and R-6, which were tested to examine the effects of a high shear force on flexural critical regions [40]. These beams were identical except for their shear span. The shear span of R-5 was $\ell/d = 2.75$; the shear span of R-6 was $\ell/d = 4.46$. Comparison of Figs. 12(a) and 12(b) show the pinching effect induced by high shear on the load-displacement relationship. This pinching effect resulted in a reduction in the energy dissipation capacity of more than 66 percent - (349 k./in.for beams R-5 vs. 738 k./in. for beam R-6). There was also a reduction in the plastic hinge rotation capacity from 0.036 radians to 0.026 radians.

Perhaps a better picture of the effect of high shear can be obtained from Fig. 13, which compares the shear force-shear distortion loading curves of beams R-5 and R-6 at comparable ductilities. As the deflection ductility of the loading reversals increased, there was increasingly more degradation in the shearing stiffness occurring in beam R-5 during the initial loading stages. Thus, there was a greater amount of shear distortion at comparable cycles. The value of average shear stiffness, K_{sh}, during the initial stage of loading to a δ/δ_y of about two was 200 k/in. for beam R-6, while shear distortion $\Delta\delta_{sh}$ at peak loading constituted about eight percent of the total tip deflection. The corresponding value for beam R-5 were 130k./in., and about 17 percent of the total deflection. After loading reached a δ/δ_y of about four, the values of K_{sh} and $\Delta\delta_{sh}/\delta$ were about 63 k./in. and 0.12, respectively, for beam R-6; and 7 k./in. and 0.37, respectively for beam R-5.

A detailed discussion of the mechanisms involved in the observed degradation, mechanical models, and a quantitative analysis of the degradation of shearing stiffness in beam R-5 are presented in Ref. 40. Results of studies at Berkeley, and other investigators reveal that:

(1) When maximum nominal shear stress induced during inelastic reversals is high in both loading direction (e.g. $5.3\sqrt{f'_c}$ (psi) or $0.44\sqrt{f'_c}$ (MPa) for beam R-5 in Fig. 12[a]) the degree of shear stiffness degradation becomes very significant. For example, the shear distortion of beam R-5 constituted about 37 percent of the tip deflection as the displacement ductility reached four. In the similar beam, R-6 (Fig. 12[b]), with a maximum nominal shear stress of $3.5\ f'_c$ (psi) ($0.30\ f'_c$ (MPa)) this value was less than 13 percent.

(2) The shear resistance in cracked R/C critical regions subjected to monotonically increasing load is developed through: (a) shear stress of

uncracked concrete; (b) aggregate interlocking and frictional resistance along cracked faces; (c) web reinforcement resistance at inclined cracks; and (d) dowel action of the main steel reinforcement. As the beam is subjected to several loading reversals, flexural and/or flexure-shear cracks may develop across the entire beam section; therefore, the shear must be resisted by web reinforcement, dowel action, and aggregate interlocking and friction. The last two resistances become less effective as the crack width increases and concrete crushes in the compression zone. As a result, large shear distortion could occur and become an important source of beam deflection as well as a significant parameter in the overall behavior of the flexural member. It should be re-emphasized, however, that this degradation occurs because of the opening of the cracks induced by yielding of the main reinforcement and is therefore a combined flexure-shear type of degradation mechanism. Because bond slippage of the main reinforcing bars can contribute significantly to the opening of flexural cracks, the deterioration observed is the result of a combined flexure-bond slippage-shear type of degradation.

(3) Photogrammetric studies conducted during the tests at Berkeley reveal that, at a ductility level of four during the initial loading state, the deformation pattern in the critical region is dominated by the shear deformation at those cracks which remain open throughout the entire beam section. For this reason this behavior has been named "Shear Sliding" and the resistance mechanism "Interface Shear Transfer" [72].

(4) The recorded shear force-shear distortion diagrams indicate that after flexural yielding occurred in both loading directions, the degradation of shear resistance and the amount of shear distortion increased with the magnitude of applied load and/or deformation as well as with each repeated cycle of reversal. The possible shear degradation mechanisms include: (a) the opening of cracks due to yielding and or slippage of the main reinforcement; (b) the spalling of the concrete cover around the periphery of the flexural critical region; (c) the degradation in the stirrup-tie anchorage due to large variations in the strains where it is crossed by inclined cracks, and/or by the splitting and spalling of the concrete cover; (d) the crushing and grinding of concrete at the crack surfaces which could lead to a less effective aggregate interlocking resistance along the open cracks; and (e) the local disruption of bond between the longitudinal steel and concrete due to the dowel action along the open cracks.

(5) The shear force-shear deformation model developed in Ref. 40 offers a reasonable prediction of the shear degradation that occurred during the initial stage of loading reversals at a beam displacement ductility ratio of one, and the first reversal at a ductility level of two. The most important parameters for determining the shear stiffness degradation appear to be the aggregate interlocking along the large cracks and the dowel action of the longitudinal steel. When loading reversals were carried out at a displacement ductility of two, the aggregate interlocking resistance could not be predicted by the analytical model since it does not account for the effect of degradation due to reversals.

As pointed out by Gergely [72], who has reviewed the problem of "Interface Shear Transfer", in beams and other R/C elements (columns and walls), several approaches have been developed for the study of cyclic interface shear transfer across open cracks in concrete. In some experimental investigations [73] the crack width was held constant during shear cycling. In another study the bars were yielded first to produce the desired initial crack width

[74]. In a third approach the bars crossing the crack were stressed in tension during shear cycling; the tension was applied to obtain the desired initial crack width [75]. An important factor affecting the behavior of interface shear transfer is the amount of steel near the crack. Reinforcement parallel and close to the crack delays or prevents shear deterioration because the concrete is confined. In addition, diagonal cracks are not allowed to propogate from the crack plane and dowel splitting is restrained. The mechanism of interface shear transfer for various types of experiments is de scribed in several papers and reports [40, 72-79]. However, the interface shear transfer mechanism under generalized (variable repeated) loading inducing large inelastic deformation still cannot be predicted accurately; more experimental studies are needed.

Experimental results have shown that the behavior of flexural critical regions under high shear stress can be significantly improved by the addition of diagonal reinforcement. This is illustrated in Fig. 14 which compares the results obtained with different types of web reinforcement [68]. The use of diagonal reinforcement is an effective means of controlling sliding shear [28, 38,39,41,68,80-83].

In a recent publication, Scribner and Wight [43] present valuable results regarding the effect of shear in beam flexural critical regions, as well as a detailed evaluation of these results and their implications in seismic resistant design. Specimens were designed with a variety of longitudinal beam reinforcement and tested using four different shear spans such that maximum shear stresses varied from $2\sqrt{f'_c}$ (psi) to $6\sqrt{f'_c}$ (psi) [$0.16\sqrt{f'_c}$ (MPa) to $0.5\sqrt{f'_c}$ (MPa)]. Half the specimens contained beam web reinforcement as specified by seismic provisions of the ACI Building code (318-71) [35] and half the specimens contained two layers of intermediate longitudinal shear reinforcement in addition to the Code-specified ties. Based on the results of these tests and in conjunction with research done by others, these authors have concluded that:

(1) The repeatability of member hysteretic behavior was related to maximum beam shear stress; (2) Intermediate longitudinal shear reinforcement provided significant increases in energy dissipation and repeatability of hysteretic response for beams with shear stresses between $3\sqrt{f'_c}$ (psi) and $6\sqrt{f'_c}$ (psi) [$0.25\sqrt{f'_c}$ (MPa) and $0.5\sqrt{f'_c}$ (MPa)]; (3) Beams with shear stresses below this range performed satisfactorily without intermediate longitudinal shear reinforcement and beams with shear stresses higher than $6\sqrt{f'_c}$ (psi) [$0.5\sqrt{f'_c}$ (MPa)] did not perform totally satisfactorily, regardless of the type of shear reinforcement used.

3.3 Seismic Behavior of R/C Coupling Beams in Shear Wall Structural Systems

In the last 10 years, significant accomplishments have been made in the study of seismic behavior of R/C coupling beams in shear wall structural systems. In Ref. 82 , Paulay has made a detailed examination of the observed performance of coupling beams in coupled shear wall structural systems. He has pointed out that these beams are often deep relative to their span. Because of this, large shear forces are generated which dominate the inelastic behavior of these beams. Typically the $V_u\,d/M_u$ ratio is equal to or less than unity. Thus the problem of these coupling beams is one of flexure with very high shear. Furthermore, the deformation capacity (ductility), the number of yielding excursions and the number of plastic rotation reversals that are demanded from these coupling beams are very large when compared with those encountered in beams of ductile-moment-resisting frames (see Section 3.1).

The performance of coupling during the 1964 Alaska [5] and the 1972 Managua earthquakes demonstrated that a conventional approach to designing and detailing these beams results in poor performance. Because of the low value of the $V_u\ d/M_u$ ratio, significant interaction between shear and flexure - usuallv disregarded in conventional design procedures - may be present.

Paulay [82] has analyzed the behavior of couplinq beams that have been designed and reinforced according to conventional procedures and those that have been designed and reinforced on the premise that the shearing force in these beams resolves itself into diagonal compression and tension forces, intersecting each other at midspan where no moment is to be resisted. This last procedure results in diagonally reinforced beams. A summary of Paulay's findings are presented below. For a more detailed discussion see Ref. 82.

3.3.1 Behavior of Conventional Reinforced Coupling Beams. This type of beam is illustrated in Fig. 15(a). Paulay has analyzed the flexural and shear behavior of reinforced coupling beams. He has also analyzed the effects of reversed cyclic loading and the effects of cracking on the stiffness of these beams. His conclusions follow.

Flexural Behavior. 1. For coupling beams with a small aspect ratio, the flexural reinforcement would experience tension over the entire span of the beam. A low stress area in the vicinity of zero bending, at midspan, does not exist and this should be noted when it is intended to splice bars near the point of contraflexure.

2. The design or analysis of the critical support sections cannot be based on the customary assumptions of doubly reinforced concrete beams. Both the top and bottom reinforcement would experience tension after diagonal cracking. For this reason the beneficial effect of the compression reinforcement ductility is not available.

3. The unexpected distribution (vis a vis, conventional theory) of the internal forces suggests that a different approach to the assessment of distortions and stiffness characteristics of coupling beams is warranted.

4. In spite of the high intensity of shearing forces, the flexural bond stresses are not likely to become critical because the rate of change of the internal tension would be considerably less than that of the bending moment.

5. Because the flexural reinforcement would be in tension over the entire clear span the length of the beam would increase with the load.

6. Because both the top and the bottom flexural bars are in tension, the internal tension resultant will be located between the levels of the top and bottom reinforcement. This suggests that such beams attempt to resist the variable moments along the span by means of near constant internal forces $T = C$ operating on a variable internal lever arm, z.

Shear Behavior. The concepts of shear resistance on which the traditional approach of designing shear reinforcement is based need to be modified for coupling beams. According to conventonal assumptions, a considerable portion of the transverse force could be resisted after diagonal cracking by arch action, because of the small shear span-to-depth ratio. However, this arch action cannot be developed in coupling beams after yielding of the web reinforcement, because the reactive shear forces at the boundaries are applied over the full depth of the beam; not in concentration form at the top and bottom surfaces. To prevent separation failure along a main diagonal (i.e. diagonal tension failure), shear must be transferred entirely by web reinforcement. Other mechanisms which might assist in shear resistance should not be relied on.

The Effect of Cracking on Stiffness. The effect of cracking on the stiffness of coupling beams may be more important in assessing the elastic response of coupled shear walls than in assessing a similar response in a ductile reinforced concrete frame. Diagonal cracking has a much larger effect on shear stiffness than flexural cracking has on flexural stiffness. Thus, diagonal cracking will have a major influence on the overall stiffness of short (small span-to-depth ratio) coupling beams in which shear distortion can dominate. Loss of stiffness due to cracking can be of the order of 85%, a quantity significant enough to be considered in the design process.

The Effects of Reversed Cyclic Loading Beyond the Elastic Limits. In conventionally reinforced coupling beams with a span-to-depth ratio of less than two, the strength and ductilities designed in earthquake resistant coupled shear walls are not likely to be attained. The load-rotation relationship shown in Fig. 15(a) indicates that only limited rotational ductility may be developed. Furthermore a sliding shear failure may result after a single large flexural yield excursion.

3.3.2 Behavior of Diagonally Reinforced Coupling Beam. The ductility and useful strength of coupling beams can be improved by placing principal reinforcement diagonally in the beam (Fig. 15[b]) instead of using the conventional steel arrangement (Fig. 15[a]). The design of such a beam can be based on the premise that shear force resolves itself into diagonal compression and tension forces intersecting at midspan where there is no moment to be resisted. Under severe seismic actions the diagonal bars can be subjected to large compression stress, and perhaps yielding that may lead to buckling, before the previously formed cracks close. Therefore it is important to have ample ties around the diagonal bars to confine the concrete inside the bars and to inhibit buckling of the diagonal steel. Figure 15(b) shows the load-rotation relationship for a beam similar to that in Fig. 15(a), except for the reinforcement arrangement. Comparison of hysteretic behavior in these two figures shows the superior response of the diagonally reinforced beams. This superior response of diagonally reinforced coupling beams has also been shown in tests carried out at the Portland Cement Association [83]. This type of reinforcement has already been used in real buildings [80].

3.4 Implications of Results Obtained to Seismic Design

In the ultimate strength design, as well as in the seismic design of R/C members, it is essential to provide sufficient shear capacity in possible hinge locations to develop required flexural and deformation capacities. In seismic resistant design, it is important to assure that such regions will not fail in shear before adequate rotation is developed at a nearly constant maximum moment. It is also important that the effect of shear degradation (pinching of the hysteretic loops) on the energy dissipation capacity of these regions be minimized. To reduce the potential shear degradation in all the critical regions, it is necessary to develop good shear resistance along the regions where large cracks might occur. The use of closely spaced stirrup-ties and supplementary ties has proven to be effective in improving the rotation and energy capacities of R/C flexural critical regions. This is so not only because more shear reinforcement is provided, preventing formation of inclined cracks, but also because such reinforcement provides better concrete confinement and provides more effective lateral support for the longitudinal compression steel. In very short beams, however, a major crack transversing the whole section can develop between two adjacent vertical ties. Consequently, these ties cannot function as shear reinforcement. In this case, the use of inclined reinforcing bars appears to be a practical solution.

4. SEISMIC BEHAVIOR OF STRUCTURAL CONCRETE COLUMNS

4.1 General

In several structural systems, and particularly in moment-resisting frames, there are columns which are subjected to very low axial forces; under lateral load the behavior of their critical regions is controlled by flexure. There is very little difference between the behavior of these columns and that of the beams, except that the columns are usually subjected to higher shear than the beams because they are shorter. Therefore, these columns can be classified with the beams under the general denomination of flexural members. This has be recognized by certain seismic codes like the ACI 318-71 and 318-77 [35] which specify that columns shall be designed and detailed in accordance with requirements for flexural members when: the maximum factored axial load P_e is not greater than $0.4\phi\ P_b$, where ϕ is the strength reduction factor and P_b is the nominal axial load strength at balanced strain conditions.

This section of the report emphasizes behavior of concrete columns in moment-resisting space frames, which are ordinarily reinforced and whose critical regions are subjected to significant axial forces. In seismic-resistant analysis and design, a frame structure can be modelled as a planar frame and subjected independently to the horizontal component of the ground motion acting in the plane of the frame [36]. Therefore at first discussion will be limited to the behavior of columns loaded in one of the principle axes (1D). Later, the importance of three dimensional loading (3D) and particularly biaxial loading will be reviewed.

4.2 Columns in 1D R/C Moment-Resisting Frames: Ordinarily Reinforced and Subjected to High Shear Forces and Significant Axial Forces

4.2.1 Experimental Studies. A review of the work carried out up to 1972 is given in Ref. 16. Since 1972 there have been many investigations of the behavior of columns subjected to seismic excitations. Work carried out between 1972 and 1977 has been reviewed in a series of papers [25, 28, 47, 72, 84] presented at the Workshop on Earthquake-Resistant Reinforced Concrete Building Construction [17]. Recent work on columns is discussed also in Refs. 85-92. Despite these studies and some significant improvements in understanding behavior of columns, some of the problems whose solutions were unknown in 1972 [16] still have not been solved. For example, some very few studies have been conducted on the behavior of columns subjected to combinations of tension and bending induced by generalized cyclic excitations. Such studies would be especially relevant because during an extreme earthquake the axial forces in columns can be lowered to values that can crack the concrete throughout the sections, especially when there are large creep and shrinkage effects. Research in this area is urgently needed because the importance of interface shear transfer in members in tension has not been examined as pointed out by Gergely [72].

The importance of such studies is illustrated by a simple example. A 16 in. by 16 in. (400 mm by 400 mm) column with eight #9 bars, when subjected to pure tension such that the stress level in the bars approach yield, will develop crack widths of about 0.02 in. (0.5 mm). Such cracks usually form at ties and therefore the ties closest to a crack would be one tie spacing away. In the absence of transverse reinforcement close to the crack, the reversed cyclic interface shear transfer capacity for such crack widths might vary from 200 to 300 psi (1.4 to 2.1 MPa); and considerable slip and crack deterioration may occur at lower stresses. The dowel capacity of the bars is equivalent to only about to 50 psi (0.35 MPa), thus sliding shear distress is possible in columns subjected to tension. For a given interstory relative displacement of, say 0.5 in. (13 mm), the compression columns would also have to distort the same amount and could undergo damaging sliding shear displacements. A slip of less than about 0.05 in. (13 mm) is already harmful; such a slip is quite conceivable considering the relative magnitudes of approximate sliding shear and column lateral stiffnesses [72].

All the available data have been obtained under excitations which produce compressive axial forces, shear, and bending in only one main plane of the element. Since columns are usually subjected to biaxial shear and bending, there is an urgent need for experimental and analytical studies of the inelastic behavior of columns under the combined effects of axial force and biaxial shear and bending. Also, during extreme earthquakes tension forces can be developed due to overturning moments and the vertical component of acceleration. Therefore it is of paramount importance to carry out studies on columns in which the axial force is varied from compression to tension. To the best of the author's knowledge, the only experimental work on the effect of varying axial force in columns has been carried out in Japan [142] and more recently at the University of Texas. These studies will be discussed later.

The columns are still the elements most susceptible to failure in destructive seismic ground motions. This has been demonstrated by inspection of damage in many recent earthquakes. In Ref. 93 Aoyama discusses the causes of shear failure of columns and countermeasures taken in Japan. The causes of such failure can be found, the author believes by: (1) Analyzing present methods of evaluating column action during severe earthquakes, and estimating the range of demands placed on columns; (2) Studying the sensitivity of the nominal unit shear stress to the main factors involved in its computation; (3) Comparing this sensitivity with the values specified by present seismic codes.

Park [25] and Paulay [47] discuss the problems encountered in estimating the column actions and therefore in formulating a design procedure that will give an acceptable degree of protection against undesirable behavior.

To illustrate the sensitivity of the nominal unit shear stress to the main factors involved in its computation the following equation compares the probable realistic value of the maximum nominal unit shear shear stress, v^R_{max}, with the value given by ACI or UBC codes v^C_u [35, 36].

$$\frac{v^R_{max}}{v^C_u} = \frac{M^R_{max}}{M^C_u} \; \frac{L^R_c}{L^C_c} \; \frac{\Phi^C_V}{\Phi^C_{(M,P)}} \; \frac{(b_w d)^R}{(b_w d)^C} \qquad (1)$$

where:

M^R_{max} = Maximum moment that can be developed in the real column

M_u^C = Ultimate moment capacity computed according to code provisions

L_c^R = Actual distance between critical regions where M_{max}^R developed

L_c^C = Nominal clear height of the columns assumed in the analysis and design of column

Φ_V^C = Code strength reduction factor for shear

$\Phi_{(M,P)}^C$ = Code strength reduction factor for columns

$(b_w d)^R$ = Actual size of columns

$(b_w d)^C$ = Assumed size of columns used in code equations for estimating v_c^C

In eq. 1, the M_{max}^R / M_u^C can be expressed as:

$$\frac{M_{max}^R}{M_u^C} = \frac{f_{s\ max}^R}{f_y^C} \; F\left[\frac{P^R \ \& \ M_{max}^R}{P^C \ \& \ M_u^C}\right] \qquad (2)$$

where:

$f_{s\ max}^R$ = Maximum steel stress that can be generated

f_y^C = Code specified yield strength

P^R = Actual axial load action on section where M_{max}^R is generated

P^C = Axial load estimated based on code force acting on section where M_u^C is computed

Considering that $f_{s\ max}^R$ can be up to twice f_y^c and $F\left[\frac{P^R \ \& \ M_{max}^R}{P^C \ \& \ M_u^C}\right]$ can also be considerably larger than one, either because P^R is a compression force smaller than the P corresponding to the balanced point but larger than the assumed or estimated P^C or because P^R can be a tension force decreasing the actual shear resistance, the value of the M_{max}^R /M_u^C can be larger than two. Additionally, because of the actual effects of lap splicing and/or nonstructural elements, the value of L_c^R / L_c^C could be larger than one, the Φ_V^C / $\Phi_{(M,P)}^C$ is 0.85/0.70 (ACI 318-71), and $(b_w d)^R$ / $(b_w d)^C$ can be larger than one.

Therefore the real nominal unit shear stress, v_{max}, can be four or more times than the value obtained using code procedure v_u^C ; the value for which the shear reinforcement was designed. Thus it is not surprising that many of the frame failures observed have been due to shear failure of the columns. Therefore there is a need to conduct statistical studies of the value of v_{max}^R / v_u^C in existing buildings.

4.2.2 Experimental Research and Development in Japan. Ohmori in Ref. 84 points out studies in Japan where the behavior of columns was investigated in order to improve the design and construction of real R/C structures. Among the important experimental findings of the research, the following deserve special mention.

Newly Developed Transverse Reinforcements--The three types of lateral reinforcement shown in Fig. 16 (a), typical of present construction, were tested, considering three levels of reinforcement ratio for each type. The results reviewed in Ref. 16 and summarized in Fig. 16(b) show the advantages of tied (type A) and spiral (type S) columns when compared with hooped columns (type J). However construction of tied columns presents some difficulties. Therefore, new types of transverse reinforcements that could offer good confinement and could be easily fabricated were sought [142]. A combination of spiral and hoop reinforcements was developed which was named the KS type. It showed ductile and stable hysteretic behavior similar to that observed for tied columns (Fig. 16[b]). Figure 16(c) shows the different types studied and Fig. 16(d) shows the results obtained. It is clear that the KS type columns showed the most stable and ductile behavior. The other two types which showed good behavior for the same reinforcement ratio, were the spirals (S) and the tied (T) arrangements.

Splices of Large Size Re-Bars--In the lower story columns of tall buildings it becomes necessary to use large size rebars. In this case the use of lap splices offers serious difficulties. Various types of welded and sleeve joints were tested. Among the most effective in the sleeve category were the squeezed joint, and the Caldwell joint [94].

Very little information is available on the behavior of lapped splices. As pointed out by Gergely [72] impact type tests of splices showed an increase in splice capacity and stirrups enhance the toughness and ductility of splices. Research is needed on the behavior of lapped splices and mechanical splices at high-level load reversals.

4.2.3 Concluding Remarks Regarding Experimental Studies. From analysis of results obtained in different investigations up to date it can be concluded that:

(1) Short R/C columns, if designed and detailed to satisfy code recommendations for ductile moment-resisting frames [36] can develop moderate inelastic deformation prior to either a brittle shear failure or significant shear degradation, when subjected to high constant axial loads and to cyclic shear reversals. The word moderate should be emphasized. It is felt that the inelastic deformation capacities found in the investigations (particularly in Ref. 88) would prove adequate when compared to the magnitude and nature of inelastic deformation demands that may be expected for columns that are components of frame systems designed on the basis of a weak girder-strong

column philosophy. However, these deformation capacities may be insufficient when compared to the magnitude and nature of deformation demands that may be expected in frames designed with soft stories. Furthermore, the above observations are valid for cases where there is essentially no fluctuation in axial force. The change from a ductile shear-compression failure mode in columns with certain axial compressive force to a brittle diagonal tension mode in similar columns in which the axial load decreased, suggests the need to investigate the inelastic behavior of short columns in which the axial force varies. The axial force should be varied with shear reversal from a maximum compression to either a tension value or a smaller compression.

(2) A comparison of analytical and experimental shear strengths indicates that code shear capacities are adequate if actual mechanical characteristics are used. However, if the expected inelastic deformations are higher than those used in the tests, these code provisions may be inadequate. The concrete degradation associated with large inelastic cyclic deformations will result in an entirely different state than that on which the code recommendations are based. To develop such large deformation and still maintain shear strength, the contribution of concrete should be ignored unless the core concrete can be kept effectively confined, even under the largest deformation.

(3) Comparison of the behavior of columns subjected to different deformation histories demonstrates that cyclic deformation reduces the maximum inelastic deformation a member can experience in a given direction. This fact should be kept in mind when design is controlled by inelastic deformation demands. It will be necessary to specify not only the deformation level that is expected, but also the number and type of reversals (partial, full) expected. The magnitude of the nominal shear stresses developed in some of the columns tested show that moderate ductile behavior and high shear stresses are compatible. However, it is necessary to provide sufficient and properly detailed transverse reinforcement.

(4) A comparison of the behavior of columns with different types of transverse reinforcement indicates that the circular spiral is more effective in maintaining a member's shear strength. Its continuity and relatively close spacing provide excellent confinement for the core concrete and restrain the width of inclined shear cracks. However, the close spacing of the spiral, and the fact that it is responsible for significant spalling through the height o of the column, reduces the area of concrete in concrete with the longitudinal reinforcement and thus contributes to bond deterioration along this reinforcement.

4.2.4 Analytical Prediction of the Hysteretic Behavior of Columns. Many attempts have been made to predict the hysteretic behavior of columns, starting from the mechanical characteristics of the materials used and following the classical approach of continuous mechanics. In Ref. 88 Zagajeski and Bertero discuss different methods and models that have been used, and the difficulties encountered in predicting the hysteretic behavior. Perhaps an easier approach is to directly model the load-deformation relationship as was done by Atalay and Penzien for flexural members subjected to high shear [95, 96]. This was done also by others [98] using a degrading trilinear model for the restoring force-deformation characteristics of reinforced concrete structures failing primarily in flexure. Jirsa in Refs. 28 and 97 reviewed the analytical work done in modeling behavior of columns and classified these different models in two categories: "conceptual model" and "element or filament model". An example of good agreement obtained by using conceptual models is shown in Fig. 17(a).

In 1977, after reviewing analytical studies, Gergely [72] concluded:

(1) Many researchers have used various types of idealizations and hysteresis rules in nonlinear analyses and have shown that good results can be obtained when the idealizations directly correspond to the system being modeled. In most cases, however, not all modes of stiffness deterioration were included in the analysis and in the corresponding tests. Significant advances have been made in system identification techniques that allow the determination of stiffness properties from test results, or enable linearization of non-linear systems. Most nonlinear analyses are too complex for design use but they are helpful in identifying the effects of various factors as well as in aiding in the planning of test programs.

(2) Many factors affecting nonlinear response have not yet been isolated or studied sufficiently. Therefore, most analyses are reasonably accurate only for the test program for which they were derived. If other factors modify the behavior or if a different type of loading is applied, the agreement between analysis and test is generally poor, especially after two or more load cycles.

4.3 Hysteretic Behavior of Columns under Three Dimensional Loading

An earthquake ground motion at the foundation of a structure has six simultaneously acting components: three translational and three rotational. Thus, the columns in a space frame are subjected to three dimensional (3D) loading components, which will vary with time during dynamic response to the ground motion. This is particularly true in the case of exterior columns in a space frame. In the case of interior columns the variation of axial force during the earthquake might not be important and therefore, although there is a state of 3D-loading, only the biaxial bending and shear will vary with time. It is common to refer to this as a case of biaxial bending or two dimensional (2D) behavior, although strictly speaking a 3D-state of loading exists even for a 2D ground motion.

In 1972 the author pointed out the lack of data regarding the 3D behavior of columns [16]. As a consequence of damage from the 1968 Tokachioki and 1971 San Fernando Earthquakes, several studies were conducted to see if it was possible to analytically predict such damage. Most of these analytical studies were based on the conventional planar behavior of the structural elements. Since these studies did not sucessfully justify the observed damage, and since there was evidence of biaxial bendings in certain columns, particularly in the case of the main buildings of the Olive View Hospital, analytical studies of the effects of 2D ground motions were started.

4.3.1 Analytical Studies on the Effects of 2D Ground Motions and Biaxial Loading on Columns. Japanese and American researchers have conducted a series of analytical studies on the effects of 2D ground motions on columns. The analytical work of Takizawa and his associates in Japan [99] and Pecknold and his associates in the U.S. [100] deserves special mention.

In Ref. 99 Takizawa concludes that "The margin of safety against collapse of R/C structures is very small when the effects of biaxial response, deteriorating ductility, and gravity are all combined. In Ref. 100 Pecknold and Suharwardy review the analytical work conducted until 1977 and summarize the findings as follows:

"2D excitation of single mass systems produces a greater period shift, which in turn can lead to larger displacement response, depending to some extent on the initial system period. Gravity loads acting through the increased lateral displacements may cause collapse. Although details of input motion and shape of hysteresis curve play a role, they do not appear to decisively influence the general trends. The combined effect of correlation of the orthogonal components of response and of inelastic interaction generally appears to increase with relative strength of the excitation. 2D ductilities about twice as large as 1D ductilities are typical at 1D ductilities of about 5 or more.

Since the effect of gravity load is consistent, an examination of responses without the P-δ effect is sufficient to indicate possible problems. Two criteria are useful for this prupose: 1D ductility demand and system period. The most important indicator is the 1D ductility demand calculated from a one-dimensional inelastic response analysis. If the system strength is sufficient to restrict the 1D ductility demand to about two, no difficulties should occur. In conjunction with this, however, the system period should be taken into account, since the consequences of a slight underdesign are more serious for short period (stiff) systems than for long period (soft) systems.

Frames resisting seismic loads in both horizontal directions should be designed so that column deformations do not substantially exceed "yield". An important factor not accounted for by response studies of single mass systems is the distribution of inelastic deformation between girders and columns in space frames, This distribution is different for 2D motion than for 1D motion, since columns may yield sooner in 2D motion. The few results available for multi-story structures indicate that 2D motion increases column response ductility demand and decreases girder response ductility demand. While a varying axial load does produce large changes in the lateral restoring force-deformation characteristics of a single column, when these characteristics are averaged over several columns in a story, the effect on the total lateral force-deformation resistance curve for the story appears to be slight. The influence of ground motion characteristics should be more thoroughly explored. Besides duration and general intensity level of the excitation, the relative strength of all components is important. Extensive work remains to be done along these lines."

Jirsa, et al. presents a thorough review not only of analytical work in this field but also of the experimental studies that have been conducted until 1978 [97]. In reviewing the analytical work, Jirsa, et al., classified the proposed models in two categories: conceptual; and element or filament models. They then summarized the models, applications, advantages, and disadvantates. One of the main problems in conducting experimental studies is the selection of realistic loading histories. The problems discussed for 1D models are increased considerably because of the many possible combinations of the path of the two components.

4.3.2 Experimental Studies. These studies can be classified as studies of flexural behavior and shear behavior [97].

4.3.2.1 Flexural Behavior. Takizawa and Aoyama [98] conducted some experiments and compared their test results with analytically predicted values on a conceptual model. Measured and analytically predicted response for unidirectional and 2D loading histories are shown in Fig. 17: The measured and analytical responses for the square (or diamond) loading history (Fig. 17[b]) are shown in Fig. 17(c). Note that the general shape of the measured curves is predicted by the analytical procedure. However, the magnitude of forces tends to differ, particularly at the largest deformation level, where the measured forces were considerably less than the predicted ones. This is apparent in the plot of the experimental and analytical force orbits shown in Fig. 17(d). The force orbit represents the locus of forces in the principal directions produced by the deflection orbit shown in Fig. 17(b).

Takiguchi and Kokusho [101], presented a summary of results from 26 specimens subjected to biaxial bending moments. The specimens were small, 10 cm. and 15 cm, square cross sections. The experimental results were compared with analytically predicted values using a finite filament model, and good agreement was found. Takiguchi and Kokusho concluded that "The influence of bending moment about one axis due to dead load on hysteretic characteristics about the other axis should be taken into consideration when conventional seismic resistant design methods (i.e. methods in which lateral forces are applied independently in two directional orthogonal to each other) are used for reinforced concrete columns."

Okada, Seki, and Asai [102] compared experimental results with the analytically predicted ones using a finite element model, and concluded that their analytical model simulated behavior reasonably well. As the severity of the 2D loading increased, the measured response clearly indicated the deterioration of strength and deformability of the columns.

Effect of Axial Load on Flexural Behavior As pointed out by Jirsa [97], the effect of axial load in the above studies was not significant. However, in the specimens tested the axial load was small or zero and remained constant throughout the 2D moment or lateral loading history.

4.3.2.2 Shear Behavior under 2D Loading. From the point of view of seismic resistant design, the ideal frame system would be one of which column hinging is prevented. This is not usually economically feasible. However, an acceptable degree of protection against premature yielding and excessive hinging should be attempted [20, 25, 47]. This design philosophy implicitly requires that shear failure be prevented or delayed so that the column may dissipate, by flexure yielding, an energy larger than that demanded by the most severe earthquake. This degree of protection against shear is not always easily achieved in practice, when columns are loaded in 2D.

As pointed out by Park [25], "The diagonal shear force resulting from biaxial bending in two-way frames due to concurrent seismic loading should be considered in design". The shear strength of rectangular column sections loaded along a diagonal has received little attention in the past. Tests have been conducted recently in New Zealand [103] on four reinforced concrete members with a 16 in. (406 mm) square section subjected to uniaxial or diagonal shear force and flexure with no axial load applied. Two arrangements of overlapping hoops were used. The difference between the diagonal shear strength and the uniaxial shear strength of identical specimens was zero for one pair and 3% for the other pair. The result is not surprising, since, although for diagonal shear the component of transverse bar forces in the direction of the shear force is smaller, the diagonal tension crack has a greater projected length and therefore intercepts more transverse bars: these effects compensate each other [25].

Jirsa and his associates have started an extensive experimental program on the effect of high shear on columns [97, 104]. The primary variable is the loading history. The geometry of the specimens is the same for all tests. The column is a stiff element (12 in. square, 36 in. long) framing into fixed ends representing a stiff floor system. Two series of tests, (one with no axial force and the other with varying axial load), have already been carried out and reported by Jirsa and his associates [97, 104].

<u>2D Behavior - No Axial Load</u> Figures 18(a) and 18(b) compare the lateral force-deformation curves for two tests. In one test, the load history was applied in 1D, in the other it was applied in 2D following a square deflection path. The force-deformation relationships are shown for a principal axis of the column. Such a comparison indicates a severe reduction of capacity due to prior or simultaneous loading in the orthogonal direction. This is shown by the force orbit in Fig. 18(c), for the specimen subjected to a square load path. If, under 2D loading, the resultant force ($V_R = V_N^2 + V_{EW}^2$) is plotted against the resultant deformation or the radial deformation from the original position ($\Delta_R = \Delta_{NS}^2 + \Delta_{EW}^2$), differences between 1D and 2D response are not as large (Fig. 18[d]). Jirsa and his associates [97] pointed out that, while a great deal of additional testing will be needed to qualify the response, results to date indicate that 2D response "may be well correlated to resultant force-resultant deformation behavior regardless of the deformation path".

Otani, of the University of Toronto, Canada has recently started an experimental program to investigate the effects of 2D deformation on columns. He has reported the results from tests of two relatively slender columns (12 x 12 x 60 in.) [105]. Because of early fracture of the longitudinal reinforcement at the welding in a critical region, no data has been obtained under cyclic loading requiring large inelastic deformations. From the results obtained, Otani concluded that:

(a) An effect of biaxial lateral load reversals on the behavior of reinforced concrete columns was evident prior to the tensile yielding of longitudinal reinforcement;

(b) The effect of biaxial lateral load reversals was relatively small, in the specimens tested, after the tensile yielding of longitudinal reinforcement;

<u>3D Behavior with Varying Axial Load</u> As "mentioned above, more research on the effects of varying axial load on column behavior is needed." (research to date has been reported by Ohmori [84], Kokusho, et al. [106, 107], and Jirsa and associates [97]. The experiments in Japan were conducted under uniaxial bending; the work done by Jirsa was under biaxial bending. Jirsa concluded that while constant compressive loads had a slight influence, constant tensile loads had a greater influence on columns subjected to biaxial bending in comparison to an axially unloaded column subjected to biaxial bending. In particular, under cyclic biaxial bending, compressive loads increased the shear capacity slightly and tensile loads substantially reduced the stiffness of the column and the shear capacity at low load. However this reduced shear capacity did not deteriorate, even under large lateral deformation. Additional tests were conducted with 2D lateral loadings and axial load variation; however, the trends are not significantly different from those under constant tension or compression. Axial loads appear to have an influence on response only while the load is on the structure and do not influence subsequent response. This is quite different from lateral loading where loads in one direction influence subsequent response in the orthogonal

direction. It should be noted that since columns were short, the P-δ effect was negligible.

4.4 Concluding Remarks

Although there have been many advances in understanding column seismic behavior most of these have been for columns under uniaxial bending and shear. Several analytical methods and models have been suggested for the prediction of real behavior of columns. However, most of these models are reasonably accurate only for the test program for which they were derived. If other factors modify the behavior or if a different type of loading is applied, the agreement between analysis and test is often poor, especially after two or more load cycles beyond yielding of reinforcement. Furthermore, most of the models are too complex for use in analysis or design practice. However, they are needed to do parametric and sensitivity studies, thus helping to: identify the importance of various factors; and aid in the planning of comprehensive experimental programs.

The Building Research Institute in Japan recently reported the result of 140 tests carried out during 1973-1976 [108]. "This report presents some of the most comprehensive information available on the behavior of R/C elements". Thorough analyses of this and other data will permit the improvement of present seismic design of columns.

Present seismic code provisions regarding detailing of columns appears to guarantee sufficient ductility to resist moderate demand of inelastic deformations if these take place in just one of the principal planes. However, during an earthquake a column can be subjected not only to biaxial bending but also to varying axial force. Although there have been some studies of these problems, there are many more factors influencing behavior for 3D than for 1D response. Therefore, it is not surprising that few advances have been made and that some of the results obtained do not, apparently, agree. It appears that bending and shear reversals in the two lateral directions increase the degree of stiffness deterioration under uniaxial bending. There can also be a significant decrease in strength and energy dissipation if the axial force can be a tensile force when large bending and shear exists. A practical solution to minimize the problems that tensile forces can create in columns has been developed by the Kajima Corporation [84, 94]. The outer columns of the first 5 stories of a modern 18 story building were post-tensioned.

No analytical model has been developed for predicting the behavior of columns under cyclic 3D loading inducing high shear and variable axial load. Some experimental programs have been started to gather the data necessary to formulate such a model. This model is needed to carry out realistic analysis of the actual performance of real reinforced concrete structures under seismic ground motion.

5. SEISMIC BEHAVIOR OF BEAM-COLUMN JOINTS

5.1 General

Efficient seismic resistant design may be achieved through predictions, or at least visualization of the structure's mechanical behavior under the excitations which it may be subjected to during its service life. To facilitate this prediction, the ideal would be to test real structures under such excitations. Since such tests are not economically feasible, basic structural components have been investigated separately. In the case of moment-resisting frames, the beams and columns have been investigated. Significant data on behavior have been obtained, and analytical methods of prediction have been formulated and used. Therefore the questions is: Can the response of the whole structure be predicted from the independent behavior of its components? Because of the interactions between these members, it is necessary to have information regarding the behavior of certain structural subassemblages. The author has discussed this problem in detail in Refs. 16 and 26.

Figure 19 illustrates the basic subassemblages of a moment-resisting frame whose behavior is essentially planar. Note that the beam-column joints are included and that there is distinction between the exterior and the interior beam-column joints. As will be discussed later, the actual subassemblages should be 3D and should consist of at least: a column; beams framing into the columns in two orthogonal directions; the joint between these two elements; and the floor slab they support. The behavior of these subassemblages should be studied under 3D loading conditions.

Because a failure of the joint means a failure of the column, ideally the joint should be the strongest and the stiffest element of the basic subassemblage. In the past this usually has been so. Surveys of earthquake damage usually show no evidence of joint failure, except in cases of very poor detailing and construction. However, because of numerous failures in beams, and particularly in columns, recent seismic codes have much more stringent requirements regarding design and detailing of these two elements. Therefore the author believes that the joint may become the weakest link in the subassemblage. This belief has been corroborated by recent experimental results in laboratories and in the field. In many cases, although there is no visible sign of distress in the joint, it has failed internally with a loss of the required anchorage to the main reinforcing bars of the beams and/or columns.

Current knowledge of the behavior of joints subjected to forces in one principal plane of a space frame is reviewed below. Following this is a more general discussion of the problem of joints in a space frame loaded in three directions.

5.2 Beam-Column Joints in Planar Frame System

In Ref. 16, the author made the following observations:

(1) Types of specimen: Subassemblages like those indicated in Fig. 19(a), where part of the floor slab is reproduced and gravity forces are applied through this slab, should be tested.

(2) Method of Testing: All tests must have a standard loading arrangement and sequence. The proper loading sequence can only be obtained by integrating analytical and experimental studies. The usual sequence of

loading is that of gradually increasing the peak value of the load or deformation (Fig. 5[b]). This method can be conservative or not, depending on what element controls the behavior of the subassemblage. If the behavior is controlled by the beam or column, this loading sequence will give upper bounds for strength and energy absorbed and dissipated. If a lower bound is desired, it is best to use a sequence starting with large peak load and deformation cycles. However, if a weak panel zone controls the behavior, the gradually increasing load sequence will give a lower (conservative) bound. Another important consideration is the magnitude of peak deformations in each cycle and the number of cycles to which a specimen should be subjected. The magnitude of the peak deformation and number of cycles to which the specimen should be subjected depends on the type of construction as well as on the type of earthquake. Again, only integrated analytical and experimental studies can give correct answers.

(3) Overall behavior: Stiffness degradation observed with reversal of loading is considerably larger than that obtained for critical regions under pure flexure, or bending and low shear forces. The major factors contributing to this degradation for exterior beam-column connections appear to be: diagonal cracking in the joint; crushing of the concrete around the curved portion of the anchorage of the beam-column reinforcing bars; and grinding of the concrete in these regions and along the diagonal cracking, which increases with the number of cycles. No reliable method exists to predict the quantitative effect of these factors on the joint. Thus, there is a need for research on the behavior of joints under repeated reversal cycles. Behavior of interior beam-column connections also should be more thoroughly investigated than it has been to date.

(4) Seismic design: For exterior beam-column connections, premature failure of the joint can be avoided by beams or stubs framing into all four faces of this zone. If this is not possiblem it is advisable to: (1) use large numbers of small diameter bars for beam reinforcement rather than a small number of large-diameter bars; (2) use steel with a low yielding strength and a large plastic plateau or low strain-hardening modulus of elasticity; (3) use the widest possible column to increase length of anchorage, or extend the anchorage of beam bars into a concrete stub added in the outer face of the column; (4) design the shear reinforcement of the panel zone, neglecting the concrete's contribution in resisting shear and considering the maximum actual stress that can be developed in the reinforcing bars, including strain-hardening characteristics.

Some of these observations are still valid today, and some of the problems still remain, although beam-column joints in planar frames have been studied in many countries since 1972. Experimental results up to 1977 [25,28, 39,42,47,84,85,109-116] and their implications have been discussed by Park [25], Jirsa [28], Paulay [47], and Ohmori [84] during the workshop held at Berkeley [17]. The results of these studies have been incorporated in a series of recommendations [15,16] and even in new seismic code provisions [12, 14]. Although some of these recommendations have been questioned [117,118], there is no doubt that overall they are a step toward more efficient seismic resistant joint design.

Since 1977 new studies have been conducted on beam-column joints; some of which are reported in Refs. 29, 43, and 119-124. However, all these studies have been concerned with joint strength. There has been very little improvement in predicting stiffness, stiffness deterioration, and deformation capacity of reinforced concrete joints. These problems will be discussed later.

Following is a summary of results from the above studies, and application of these results to seismic resistant design, starting with a suggested design criteria for the joints. Exterior joints are distinguished from interior joints. The summary is based on results and discussions in Refs. 25, 28, 39, 47, 119 and 124.

5.2.1 Design Criteria of Beam-Column Joints. Paulay in Ref. 47 suggested the following design criteria for joints in ductile moment-resisting space frames:

(1) The strength of a joint should not be less than the maximum strength of the weakest members it connects.

(2) The capacity of a column should not be jeopardized by possible strength degradation within the joint due to inelastic cyclic displacements of a frame.

(3) A joint should not be a prime source of energy dissipation.

(4) During moderate seismic disturbances a joint should respond within elastic limits so that no repair would be necessary for these inaccessible areas of the structure.

(5) The joint reinforcement that will ensure satisfactory performance should not present undue construction difficulties.

Although most researchers and designers agree with the above design criteria, approaches for practical design and detailing of joints vary considerably [25,28,47,116,118,124].

5.2.2 Exterior Beam-Column Joints. As Park [25] points out, an analysis of the forces acting on an external beam-column joint of a reinforced concrete frame (Fig. 20) and of the associated cracking shows that the bond conditions for the longitudinal beam and column bars are unfavorable because: (a) large steel forces need to be transferred to the concrete over relatively short lengths of bar; (b) flexural and diagonal tension cracks are present which will alternate in direction during cyclic loading; and (c) bond deterioration will occur during cyclic loading. For example, if the outer column bars are near to yielding in compression above the core and are yielding in tension below the core, approximately twice the yield force of the bar needs to be transferred to the joint core by bond over the depth of the core. The extremely high bond stresses so induced, and the anchorage forces from the beam bars, can result in vertical splitting of the concrete along the outer column bars. Thus the concrete cover over these bars spalls easily, particularly when heavy horizontal ties are used. This spalling may extend beyond the joint area, significantly reducing the flexural strength of the column, leading to hinging in the column rather than in the beam [42, 124]. Therefore, it has been suggested that the computation of column strength should be based on the strength of the column core area only [124].

If plastic hinging occurs in the beam at the column face, the anchorage of beam steel should be considered to commence within the joint core at one-half the column depth or ten bar diameters, whichever is less, from the face of the column where the steel enters. An anchorage block, in the form of a beam stub at the far face of the column where the longitudinal beam bars can be anchored (Fig. 20[c]) has been shown to improve joint performance and is being used by some designers in New Zealand. The maximum diameter of longitudinal column bars should not exceed 1/20th of the beam depth for steel with f_y = 40 ksi = 275 MPa or 1/25 of the beam depth for steel with f_y = 55 ksi = 380 MPa.

It is recommended that the nominal shear stress, v_c, carried by the concrete shear resisting mechanisms in the joint core should only be taken into account if the compressive load on the column exceeds $0.1f'_c\ A_g$. The degradation of shear carried by the concrete occurs due to repeated opening and closing of diagonal tension cracks in alternating directions and full depth cracks in the beam which results in the beam compression being transferred into the joint core by bond. The ACI 318-71 [35] assumption at 45° cracking is difficult to justify since the cracking will be parallel to the diagonal compression strut which runs from corner to corner. Hence, the design horizontal shear force in Fig. 20(a) is T-V', where T is the force in the beam bars enhanced for overstrength and V' is the column shear force. This design shear force should be resisted by the concrete, if the compressive load exceeds $0.1f'_c$, and by the force in the horizontal shear reinforcement which crosses the corner to corner crack. Vertical shear reinforcement should also exist in the form of vertical column bars around the perimeter of the column section (spacing not to exceed 6 in. (150 mm), with at least one intermediate bar between the corners. Such vertical bars are necessary to help transfer vertical shear forces. That is, four bar columns should not be used. A procedure for the design of vertical shear reinforcement has been developed [125].

The use of all these rules could lead to very conservative joint construction, but until new data is available, such requirements should not be relaxed.

5.2.3 Interior-Beam-Column Joints. Until 1972, relatively little attention was paid to interior-beam-column joints. This could have been due to the philosophy of some seismic codes regarding anchorage of the beam bars in this joint. For example, the commentary of ACI 318-71 and even ACI 318-77 [35] states, "The code does not require anchorage calculations for top and bottom reinforcement continuous through beam-column connections except for anchorage within each flexural member". The argument given is that "reverse loading tests of interior connections conforming to ACI 318-71 provisions show that the advantages of continuity offset any theoretical deficiencies in embedment length within the connections". Bertero and Popov, in Ref. 39, have questioned the soundness of this provision, because the slippage of the longitudinal beam reinforcing bars through the joint can lead to deterioration of the subassemblage's energy dissipation capacity. The importance of this degradation is illustrated in Fig. 21, which shows test results for one specimen [39,41].

Using the third-floor framing in a 20-story moment-resisting reinforced concrete building as a prototype, a half-scale subassemblage with an interior joint was designed (Fig. 21[a]). In this subassemblage, inelastic action was to develop in the beams, i.e., the design philosophy of strong columns-weak girders was followed. The beams were reinforced in exactly the same manner as beam specimens of a half-scale cantilever series of experiments (Fig. 21[b]) [40,69]. The testing arrangement for the cross-shaped specimen was such that an axial column force, as well as vertical forces at the ends of the beams, could be applied to it. Whereas the top hinge of the subassemblage remained fixed in position, the other three hinges could be displaced horizontally upon application of a horizontal force at the lower hinge. At large displacements of the lower hinge, the P-δ effect caused by the vertical load in the column was significant.

Eight similar subassemblages have been tested to date. A brief discussion of the major results follows. One of the cantilever specimens was tested under a monotonically increasing load. The lateral load-deformation relationship (H vs δ) is shown in Fig. 21(c). From this figure, it can be seen that the curve is of the softening rather than the strain-hardening type. This is as to be expected from the results obtained with the beams, Fig. 21(b) together with the added P-δ effect. The significance of the P-δ can be noted from the comparison of the two curves shown in Fig. 21(c). Besides the H-curve, there is another one for the equivalent story shear, H_{eq}, which is related to the measured story shear by the relationship, $H_{eq} = H + P\delta/h_{col}$.

In Fig. 21(d), an analytic hysteretic loop is compared with the experimental one of Fig. 21(c). The agreement for the monotonically increasing story shear is excellent. However, large discrepancies can be noted during the loading in the reverse sense and these discrepancies become greatly magnified during the initial reloading of the second cycle. The following questions therefore arise:

(1) Why is there such a sharp degradation in strength during the first reversal, after just the first loading to a displacement ductility ratio of 4.5?

(2) Why is there such a pronounced degradation in stiffness during the first reloading, after just one cycle of a full reversal?

Since nominal shear stress developed in the beams was small [on the order of $3\sqrt{f'_c}$ (psi)($0.25\sqrt{f'_c}$ (MPa))], similar to that induced in the cantilever beams of Fig. 21(b), it is clear that the observed degradation was not the result of shear in the beams. The main reason for this behavior was the slippage (pull-out) of the beams' main longitudinal reinforcement along the column joint. This is clearly shown in Fig. 21(e) where the sum of the measured pull-out and push-in of the steel bars is plotted.

The effect of repeated load reversals can be seen from the results presented in Fig. 21(f). These results were obtained from tests conducted on the specimen used in obtaining the results of Fig. 21(c) after it was repaired by injecting epoxy into the cracks. Although it was possible to achieve the strength attained during the first loading of the virgin specimen, this strength was achieved at a considerably greater deformation. During the second cycle, there was a large drop in strength from the first peak deformation reached during initial loading. As the number of cycles increased, both resistance and stiffness dropped as a result of bond deterioration along the embedment length of the beam bars.

Recently, there have been many studies of the interior joint [25,47, 114,117-123]. Many of the points made regarding exterior beam-columns apply to interior beam-column joints. In discussing ways to improve seismic behavior of interior joints, Park [25] points out that:

(1) When plastic hinging occurs in the beams at the column faces, it is recommended that the maximum diameter of longitudinal beam reinforcing

bars should not exceed 1/25th of the column depth for steel with f_y = 40 ksi = 275 MPa or 1/35th of the column depth for steel with f_y = 55 ksi = 380 MPa. The diameters of longitudinal column bars are limited as for exterior joints.

(2) The degradation of shear strength with cyclic loading occurs in the joint core for the same reason as in exterior joints. Repeated opening and closing of diagonal tension cracks, and full depth cracking in the beam at the column face, lead to a reduction in the effectiveness of the concrete diagonal compressive strut. Figure 22 illustrates the forces acting on a beam-column joint core. The forces entering the joint core are transferred across it by the diagonal compression strut (Fig. 22[b]) and by a truss mechanism involving diagonal tension and compression induced by the bond forces of the longitudinal bars (Fig. 22[c]). The shear carried by the concrete, v_c, arises mainly from the diagonal compression strut. When full depth cracking of the beam leaves the longitudinal steel as the only effective beam force transmitter, the mechanism involving truss action becomes dominant and this mechanism requires the presence of both horizontal and vertical bars to carry the diagonal tension forces across the joint core. Hence the force to be carried by the horizontal shear reinforcement increases as cyclic loading proceeds and vertical steel crossing the joint core is needed to carry the vertical forces necessary to complete the truss mechanism.

As noted by Paulay [47] although it is possible to transfer joint shear across the joint core with sufficient ties and intermediate vertical column bars, providing adequate anchorage for the main beam reinforcement presents a more difficult problem. The bond of the main beam reinforcement, anchored in the joint in the plane of the frame, can be adversely affected by the same mechanisms that are responsible for joint-core shear strength degradation: In particular by the transverse tensile strains imposed by the main reinforcement of the beams framing at right angles to the plane of the frame, and yield penetration into the joint when the inelastic regions (plastic hinges) developed adjacent to the faces of the joint. Generally, ACI 318-71 [35] development requirements cannot be satisfied for beam bars passing continuously through interior joints that are subjected to severe earthquake loading.

Excellent response to reversed cyclic loading (elimination of hysteretic pinching) was obtained at the University of Auckland [114] in specimens in which the steel forces were transferred to the core by welded bond (bearing) plates. Although this arrangement cannot be considered as a practical solution to the joint problem, the tests have clearly shown the great significance of proper anchorage within the joint.

When plastic hinges may form adjacent to columns, the diameter of the steel beam bars, passing through a joint, should not exceed the limits indicated above: 1/25th or 1/35th (depending on the grade of the steel) of the column depth in the relevant direction. If this is done, experimental evidence indicates that a large number of excursions with adequate ductility in both directions of seismic loading can be made before slippage of the bars will reduce the strength of the joint [47].

5.2.4 Elastic Joints. Two of the critical aspects of joint seismic behavior discussed above have been found to result in construction difficulties [47]. Unless the flexural tension reinforcement content in beams is kept small (i.e. less than 1.5 percent) the horizontal joint stirrup reinforcement may become so large that serious congestion of steel results. The limitation of bar size in beams, to reduce the danger of slippage, may result in the use of an excessive number of bars. Some designers have found it necessary to increase member sizes for the sake of steel placement within the joint. In spite of these measures, in conventionally reinforced joints a satisfactory safeguard does not yet appear to exist against pullout of beam bars from joints. Whenever practical, the prime cause of these difficulties, beam hinges adjacent to column faces, should be eliminated. This may be achieved by curtailing the beam reinforcement so that a deliverate weakness in flexural resistance results at a more suitable beam section. The relocated potential plastic hinge should be as near as practicable to the column face but far enough to ensure that, as a consequence of reversed cyclic loading, yield penetration will not extend to the column face. In such a beam when well designed, the steel stresses at the column face will approach but not exceed the level of nominal yield when the overstrength capacity at the relocated plastic hinges is simultaneously being developed. Therefore, if the joint core is adequately reinforced to resist horizontal and vertical joint shear force, it will remain elastic during cycling loading. This design philosophy, of moving the formation of plastic hinges from the face of the column and thereby assuring elastic joint behavior, was suggested by Bertero and Popov [38,39,68]. Experimental studies [42,119] show this to be a sound and practically feasible philosophy. Figures 23(a) and 23(b) illustrate one of the techniques used to move the beam inelastic regions (plastic hinges) away from the face of the column. (The specimen used is similar to that shown in Fig. 21[a].) The two top interior main bars of the beams were bent downward; and the two corresponding bottom bars were bent upward, intersecting 16 in. (406 mm) away from the column face. The hysteretic behavior of the specimen was excellent (see Fig. 23[c]). The hysteretic loops became pinched only after the first cycle with a full deformation reversal at displacement ductility seven. Comparison of test results of Figs. 23(c) and 21(f) shows a significant improvement achieved by moving the plastic hinge away from the column faces.

The above results have been confirmed by an experimental study carried out by Bull [126], and has been discussed by Paulay [127]. Paulay has also made recommendations which have been incorporated in the seismic provisions of the Draft New Zealand Code [14].

5.2.5 <u>Prediction of Stiffness and Energy Dissipation Capacity of Beam-Column Joints</u>. Analysis of results from investigations into the seismic hysteretic behavior of beams and beam-column subassemblages indicate that joints of R/C frames should not be considered rigid as is usually assumed. Two possible sources of deformation that may develop at the joint must be included to accurately predict the actual hysteretic behavior of the frame, particularly when large displacement ductility demands are expected. These two sources of deformation are illustrated in Fig. 24, and will be identified as the shear distortion of the joint ,γ_j, and the fixed-end rotation at the column face, θ_{FE}. Often the most important deformation is the one due to θ_{FE}. In contrast with the amount of research carried out to improve the design of beam-column joints for shear strength, very little has been conducted to improve methods of predicting stiffness, deformation capacity, and energy dissipation capacity of these joints. These mechanical characteristics are controlled by the θ_{FE}, which in turn depends on the bond-slippage characteristics of the beam bars along its embedment length at the joint.

Although excellent work has been done by several investigators on bond under generalized loading [128], to the best of the author's knowledge none of these investigations specifically addressed the problem of bond deterioration developing at the joint of an interior column. In the case of a joint in an interior column, we are dealing with bond-slippage of steel bars which are embedded in a well confined reinforced concrete but which can still be adversely affected by the mechanisms discussed in section 5.2.3. At Berkeley, there has been an investigation of the simplified problem of bond-slippage of bars embedded in well confined reinforced concrete, which simulates the conditions of a beam-column joint in a plane frame loaded laterally in its plane [129-133]. From the results of these experimental and analytical studies it has been concluded that:

(1) The assumption that beam-column joints of moment-resisting R/C frames are rigid needs to be reexamined. The main reinforcing bars of the beams do pull-out, and thereby cause beams to experience fixed-end rotation. The consequences of this behavior on the overall structural response must be examined.

(2) In the joints, it is essential to distinguish between the bond of unconfined concrete in the column cover from that of the confined core. The latter is appreciably better.

(3) Under monotonically increasing loads, when the beam main bar reaches yielding the accompanying pull-out can cause a fixed-end rotation in the order of 0.001 radians.

(4) The displacement of a bar due to monotonic loading at the column face can be estimated using simple idealizations of bond stress distribution [131]. The dependence on concrete strength, type of lugs, embedment length, concrete confinement, etc. requires further investigation.

(5) Significant bond deterioration occurs from cyclically applied load reversals, particularly when the applied stresses exceed yield.

(6) It appears that bond resistance deterioration is gradually stabilized at the value of friction between two concrete cylindrical surfaces which have a common diameter equal to the outer dimension of the bar, including the lugs.

(7) More comprehensive analytical models are required for generalized loading of a bar. (A model has been developed by Viwathanatepa [133].)

(8) The implications of the effect of θ_{FE} on the behavior of structural systems should be studied analytically. (A computer program that permits inclusion of θ_{FE} in nonlinear analysis has been developed by Soleimani [120].)

5.3 Beam-Column Joints of Space Frames Subjected to 3D Loading

As pointed out in the discussion of columns under 3D loading, the moment-resisting frame is usually a space frame having two-way frames in each joint, i.e., beams framing into the joint along the two orthogonal main axes of the structures, and subjected to ground motions with components in both directions. In spite of this, most seismic codes presently require that the joint be designed independently in each direction. Furthermore, some codes, such as ACI [35] allow the transverse reinforcement in the connection to be reduced by one-half if every beam has a width not less than one-half the column width and a depth not less than three-fourths that of the deepest beam framing into the connection. Even the new recommendations of the ACI-ASCE Committed 352 [116] for design of beam-column joints allows an increase in the shear stress carried by concrete when the joint is confined by lateral members framing into the joint. It is agreed that transverse confinement can enhance the shear capacity of the concrete. However, the question is how effective this confinement can be when critical regions (plastic hinges) are developed in the beams framing transversely into the joint.

In Ref. 25 Park has shown that if the beams in the two directions are identical and they yield simultaneously, the horizontal shear force acting along the diagonal of the joint cross section (Fig. 25) is $\sqrt{2}$ times the uniaxial shear force. However, the diagonal tension crack intersects the same number of reinforcing bars as for uniaxial shear. If these bars are parallel to the sides of the section, the diagonal component of the bar force is only $1/\sqrt{2}$ that available to resist uniaxial shear. Hence design for biaxial shear for symmetrical two-way frames can lead to approximately double the quantity of shear reinforcement required for uniaxial shear design. This can create serious practical problems, such as congestion of steel. Experimental studies of this problem are needed. Some experiments are presently being carried out at the University of Canterbury, New Zealand [121], and at the University of Texas, Austin, Texas.

5.4 Concluding Remarks

Research concluded since 1972 has resulted in significant advance in understanding the behavior of beam-column joints, leading to development in the practical design and construction of such joints. However there are some problems that still need further research and development. There is a need to study how the strength capacity of the joint can be affected by (a) the slab; (b) 3D loading; (c) the eccentricities of the elements framing into the joint; (d) the amount and type of both transverse and longitudinal reinforcement. The main parameters controlling such strength capacity should be identified. There is also an urgent need to study the joint's stiffness, the deterioration of this stiffness, and its deformation capacity and energy dissipation capacity. It is important to develop simple but reliable mathematical models of joint behavior that can be used in computational analysis to study the affect of joint behavior on seismic response of ductile moment-resisting space frames.

Until further information is available, joint design should be based on the stringent rules given above or should be based on the philosophy of keeping the joint elastic by moving potential critical regions in the beams away from the face of the columns.

6. SEISMIC BEHAVIOR OF LIGHTWEIGHT CONCRETE LINEAR STRUCTURAL ELEMENTS AND THEIR CONNECTIONS

6.1 General

There are a number of advantages to using lightweight, rather than normal weight, aggregate concrete in seismic-resistant reinforced concrete construction. One of the basic principles of such construction is to avoid use of unnecessary mass. The lower the weight of the reactive masses the lower the seismic forces that will develop as a consequence of earthquake ground motions. If one compares the standard mechanical characteristics obtained from compression test per unit weight of lightweight concrete with those of normal weight concrete (Fig. 4) or analyzes results available from experimental studies on individual structural elements there is no doubt that it would be advantageous to use lightweight aggregate concrete. Therefore, some investigators have concluded that the use of this type of concrete results in more efficient earthquake resistant construction [134, 135]. However, proper assessment of the performance of any structural system requires not only analysis of the behavior of the individual elements, but also of the assemblage of these elements. As already discussed, this is of particular importance in the case of R/C structures where connections between elements depend upon transfer of forces between the two constituent materials, reinforcing steel bars and concrete.

Current seismic codes in both the U.S. [35, 36] and Canada [136] permit the use of lightweight concrete in the construction of ductile moment-resisting space frames. The only precaution is that "the maximum specified strength for lightweight concrete shall be limited to 4000 psi (28 MPa)". Unfortunately, because of its lower modulus of elasticity, very high compressive strength concrete mixes have been used to achieve a higher degree of stiffness and this has caused some problems regarding the use of these mixes for seismic-resistant construction, particularly regarding the effectiveness of confinement, bond, and shear transfer of such concrete.

6.1.1 Confinement. References 20, 23 and 137 discuss the problems of using confined lightweight aggregate concrete for seismic construction. A summary of the observations made in these references follows.

Confinement of concrete with all types of aggregate tested was effective in developing large deformability. However, the effectiveness of concrete confinement in the performance of earthquake-resistant reinforced concrete structures should not be based only on the extent to which the deformability is increased, but also on the ability of the confined concrete to sustain large deformations without loss of strength. Therefore, confinement should also increase the compressive strength of the concrete, so that it is possible to offset the loss of strength due to the reduction of the cross-section resulting from crushing and spalling of the concrete cover.

Figure 26 shows some results of the study in Ref. 23. These results show that the conditions of increased deformability and compressive strength are satisfied to a verying extent for different concretes, and the effectiveness of confinement is highly sensitive to the type of aggregate used. The effectiveness of confinement can be characterized by two material constants, k_o and k_u, which are defined by relating the increased compressive strength f_c^*, to the confinement pressure, f_r.

The maximum compressive strength $f_c^* \max$, occurs after some strain, ε_0^*, and can be related to the unconfined compressive strength of the same concrete, f_c, and the confinement pressure as:

$$f_c^* \max = f_c + k_o f_r \tag{3}$$

At very large deformations, $\varepsilon_u^* >> \varepsilon_0^*$, the compressive strength usually decreases to a value of f_{cu}^*, and can be related to these same parameters as:

$$f_{cu}^* = f_c + k_u f_r \tag{4}$$

The confinement pressure, f_r, depends on the geometric and material characteristics of the spiral wire, and can be approximated by:

$$f_r = \frac{2A_{sp} f_s}{D_c s} = \frac{1}{2} \rho_s f_s \tag{5}$$

where ρ_s is the ratio of volume of spiral to total volume of core and f_s is the stress that had been developed by the spiral wire. Assuming that the ductile spiral wire yields when the longitudinal strain in the concrete is in the range ε_0^* to ε_u^*, and that the strain-hardening of the spiral is negligible in the range of these concrete strains: (a) f_s is equal to f_y; (b) f_r can be calculated for given values of A_{sp}, D_c, and s from Eq. 5; (c) values of k_o and k_u can be calculated from Eqs. 3 and 4, using the test results. The values for the five different concretes used in this study are shown in Table 1. Early investigators have shown that the confinement effectiveness coefficient, k, varies with lateral pressure intensity and with longitudinal strain. However, in developing the ACI criterion for spiral reinforcement (Section 10.9.2 of ACI 318-71) [35] and similar criteria which are based on the confinement of concrete, a constant value of k, usually taken as 4.0 to 4.1, has been assumed.

TABLE 1.- EFFECT OF CONFINEMENT ON COMPRESSIVE STRENGTH AND DEFORMATION OF CONCRETE.

Type of Concrete	Confinement Stress Ratio	Maximum Compression		Ultimate Compression	
		Strain Ratio	Confinement Effectiveness	Strain Ratio	Confinement Effectiveness
	(f_r/f'_c)	$(\varepsilon_0^*/\varepsilon_0)$	k_o	$(\varepsilon_u^*/\varepsilon_0)$	k_u
Normal E-5	0.13 0.32	2.8 7.8	7.0 5.0	11.5 11.5	0 3.1
Lightweight R-5	0.13 0.32	1.9 4.0	4.4 2.0	8.7 6.7	-0.5 2.0
B-5	0.13 0.32	1.35 1.85	3.9 1.0	10.6 8.6	0 0.9
R-3	0.11 0.24	1.8 5.9	2.7 2.5	8.9 8.9	-1.0 2.0
B-3	0.11 0.24	1.7 8.0	1.35 2.1	11.6 9.0	0 2.1

As shown in Table 1, the values of k for normal weight aggregate concrete vary in the range of 0 to 7.0. For the two lateral pressures (0.13 f_c and 0.32 f_c), values of k_o at maximum compression are 7.0 and 5.0 respectively, and values of k_u at ultimate strength are 0 and 3.1 respectively. Based on these values, and noting from Fig. 26 that concrete behaves in a relatively ductile manner throughout a significant range of strains, a constant value of k = 4.0 may be justified for normal weight concretes such as E-5, particularly in the case of $f_r = 0.32(f_c)_{10}$.

For lightweight concretes B-3, B-5, R-3, and R-5, the values of k vary in the range of -1.0 to 4.4. Negative values of k_u indicate that compressive failure in the confined concrete may occur at values below the compressive strength of unconfined concrete. For the two lateral pressures ($f_r \simeq 0.1\ f'_c$ and $f_r \simeq 0.3\ f'_c$), values for k_o at maximum compression range from 1.0 to 4.4 and values for k_u at ultimate range from -1.0 to 2.1. Based on these results, a value of k in the range of 1.0 to 2.0 should be taken in developing design criteria based on the increase in strength due to confinement of lightweight concrete. Therefore, the amount of spiral steel required in a column of lightweight aggregate concrete will be 2 to 4 times greater than that currently prescribed by the ACI Code [35]. Because of the geometric limitations introduced by the size of the spiral wire and the minimum spacing, it would be virtually impossible to produce a spiral which would also allow normal placing of concrete.

The effect of the variable coefficient, k, is illustrated in Fig. 27. In this figure, the loss of the axial load carrying capacity for spirally reinforced concrete columns due to spalling is plotted against k, assuming that the spiral reinforcement was designed in accordance with the ACI criterion [35]. This loss of capacity is expressed as a ratio and derived as:

$$\text{Loss} = 0.85f'_c(A_g - A_c) - kf_rA_c\ ;$$

and using Eq. 5

$$\text{Loss} = 0.85f'_c(A_g - A_c) - 0.5k\rho_s f_s A_c \qquad (6)$$

According to the ACI criterion, $\rho_s = 0.425\ [(A_g/A_c) - 1](f'_c/f_s)$. By substituting this equation into the above, and dividing by $0.85f'_cA_g$, the following ratio is obtained

$$\frac{\text{Loss}}{0.85f'_c\ A_g} = (1 - \frac{A_c}{A_g}) - 0.25k(1 - \frac{A_c}{A_g}) \qquad (7)$$

Typical values of A_c/A_g (where A_c is the area of core and A_g is the gross area) for spirally reinforced square columns vary from approximately 0.4 to 0.6. For round columns this ratio varies from approximately 0.5 to 0.7. The loss ratio for typical values of A_c/A_g is plotted in Fig. 27, which illustrates the significant losses that can occur due to k values lower than 4.

Most of the recent suggestions and requirements for improved design of earthquake-resistant reinforced concrete structures rely on the beneficial effects of confinement on concrete behavior. Thus it is important to analyze the implications of the results summarized above [23] with regard to seismic behavior of concrete structures. Some observations obtained from such analyses follow [137].

1. Confinement of concrete with all types of aggregates is effective in developing large deformability, i.e. large ultimate strains. This characteristic is the major factor in the improved performance of elements with spirally confined concrete, as it compensates for some of the losses in strength and stiffness of concrete under cyclic loading.

2. The increase in compressive strength due to confinement is about twice as great for normal weight concrete as for lightweight concrete. Therefore, one should be cautious in using equations from tests on normal weight aggregate concrete to predict behavior in lightweight concrete.

3. The low effectiveness of confinement in some concretes may lead to significant losses in compression capacity when spalling occurs. This is of utmost importance in the seismic design of column elements, since these elements should be able at all times to resist the effects of gravity loads and overturning moments.

These conclusions have been confirmed in a recent experimental study[138].

6.1.2 The Bond and Shear Transfer Problems. Recent bond tests performed at Berkeley [129-133], on specimens simulating the conditions of an interior beam-column joint, demonstrated that the deterioration of bond in lightweight concrete occurred under smaller steel strains than in normal weight concrete [132].

In the case of flexural critical regions under high shear, one of the main factors controlling the degradation of stiffness is shear transfer along the cracks. Mattock has conducted a series of studies on the problem of shear transfer along cracked concrete[78, 79, 139]. Based on test data obtained in these studies, Mattock has concluded that "the shear transfer behavior of initially cracked all lightweight concrete is more brittle than that of sanded lightweight or sand and gravel concrete," and that "shear transfer behavior across a crack becomes more brittle as the concrete strength increases".

The above studies examined the three basic problems in the behavior of lightweight concrete - the effectiveness of confinement, bond, and shear transfer. The studies showed that, for seismic-resistant construction, lightweight concrete has certain deficiencies in addition to its low modulus of elasticity. These deficiencies indicate a need for further studies in order to properly modify the proportioning and detailing rules obtained from and used for members cast of normal weight concrete, so that these rules can be applied to lightweight concrete.

6.2 Behavior of Linear Elements and their Subassemblages.

6.2.1 Studies of Beam Behavior. Very few studies have been reported on lightweight concrete beams subjected to seismic action. Mihai, et al. [140] have carried out some tests on lightweight aggregate concrete beams columns and their connections and have concluded that:

"Generally the ductility of bending members of granulite lightweight concrete is 15-40% greater in comparison with that of similar members of heavy concrete. In the case of members subjected to compression with bending, the ductility factors are close for the similar members made of heavy concrete, and lightweight concrete. With a proper detailing conception, the joints realized with lightweight concrete are more ductile, with 15-25% increases, in comparison with heavy concrete ones. The more elastic and also more breakable behavior of lightweight concrete, requires detailed and careful experimental and theoretical research for all types of granulite material".

Because of insufficient detail it is difficult to judge what definition of ductility the author of Ref. 140 has used.

6.2.2 Studies of Column Behavior. Experimental studies show it is possible to achieve good ductile behavior by properly confining lightweight concrete with spiral or closely spaced and carefully detailed rectangular hoops and ties [140, 141, 142]. However the only comparison available between similar specimens cast of lightweight and normal aggregate concrete show better strength, stiffness and ductility for the normal aggregate concrete [142]. In Section 6.1, some drawbacks of the use of lightweight aggregate concrete were discussed. In addition, lightweight concrete has a higher rate of creep than normal weight concrete. Therefore, serious questions remain regarding the use of lightweight concrete in columns, especially in tall frame buildings. In the lower stories of buildings, high axial loads caused by gravity loads can cause : a higher rate of creep and larger P-δ effects of lightweight than for normal weight concrete, due to the lower stiffness of lightweight concrete. Comprehensive experiments are needed to find the role of these effects on the hysteretic behavior of lightweight concrete columns.

6.2.3 Subassemblage Behavior. As discussed in Section 6.1, proper assessment of the performance of any structural system requires studying the behavior of the system's basic subassemblages. Studies were conducted at Berkeley [29] of the behavior of basic subassemblages of a ductile moment-resistant space frame (DMRSF) built of lightweight aggregate concrete. The completed study had two main objectives. The first was to study the behavior of a DMRSF subassemblage constructed of lightweight aggregate concrete under earthquake-like load conditions and to compare such behavior to that observed under monotonic loading, paying particular attention to the effects of bond degradation in the joint region. The second objective was to compare the performance of lightweight R/C subassemblages to that of previously tested normal weight subassemblages for both monotonic and cyclic loadings. Figure 21(a) shows the specimens which were used: half-scale models of interior beam-column subassemblages from the third floor of a twenty-story office building. A summary of the results of these tests follows.

Figure 28 compares the behavior of lightweight aggregate specimens (BC7 and BC8) with that of normal weight subassemblages (BC3 and BC4) of similar concrete strength and steel yield strength subjected to similar applied displacement programs. Due to the greater flexibility of lightweight concrete, ductility, μ_δ, rather than absoluted displacement was used as the base of comparison.

Monotonic Loading - From analysis of the curves shown in Fig. 28(a) it is clear that the overall behavior of the lightweight concrete was very similar to that of normal weight concrete. Furthermore, the contribution of the fixed end rotation θ_{FE}, due to slippage of the beam main bars along the joint, to the lateral displacement, δ, was approximately the same for BC4 and BC7. However, the initial stiffness, which is highly dependent on the material stiffness of the concrete, was 52 percent higher in BC4. This was in agreement with the relative moduli of elasticity of the two specimens, as BC4 had a 46 percent higher modulus of elasticity. This signifies that lightweight R/C structures will have greater nonstructural damage and higher P-δ moments for the same displacement ductility.

Cyclic Loading - The performance of the normal and lightweight concrete specimens under incrementally increasing cyclic loading differed significantly as shown in Fig. 28(b). Specimen BC3 reached a peak strength at LP25 (μ_δ = 3.9) and LP26 (μ_δ = -4.2) while the strength of specimen BC8 peaked much earlier; at LP17 (μ_δ = 1.45) and LP18 (μ_δ = -1.75). At LP22 (μ_δ = -2.7) the capacity of BC8 was already only 70 percent of that of BC3. The difference in behavior was due to the premature total slippage of the reinforcement in specimen BC8. By LP24 (μ_δ = -2.7), the contribution of the θ_{FE} at the column face to δ was over 75 percent for BC8 while it was less than 35 percent for BC3. Total slippage of the beam bars did not occur in BC3 until LP29 (μ_δ = 5.4) when over 50 percent of δ was due to θ_{FE}. This strikingly different behavior under cyclic loading indicated that the bond within the joint deteriorates at lower μ_δ in lightweight concrete. Although the cause of this earlier deterioration is not completely understood, it is speculated that the lightweight aggretate is sheared and crushed by the lugs of the deformed bars at lower stresses, leading to earlier bond deterioration. Propogation of cracks formed by the action of the lugs might also be affected by the type of aggregate used.

6.3 Concluding Remarks. From the available information, particularly from results of studies carried out at Berkeley, the following observations can be made. Because of the relatively meager data available, these observations are of a preliminary nature.

1. Individual lightweight aggregate members have a similar hysteretic behavior to normal weight aggregate members of similar strength. The only remarkable difference is the lower stiffness of lightweight concrete, which means larger deformation is needed to develop the same displacement ductility.

2. Beam-column subassemblages subjected to monotonic loading show that a displacement ductility (μ_δ) in excess of 5 can be achieved without a decrease in resistance. Behavior is very similar to that of the normal weight specimen. For the same ductility displacement ratio the total displacement and the story drift is greater than that of the normal weight specimen, causing larger P-δ moments.

3. Under cyclic loading, the behavior of beam-column subassemblages cast of lightweight aggregate concrete is drastically different than that under monotonic loading, due to earlier slippage of the beam reinforcement through the joint. Yielding of this reinforcement accelerates bond deterioration and therefore slippage.

4. Under cyclic loading, the energy dissipated by beam-column subassemblages is smaller than that of similar normal weight subassemblages. The main reason for this was that total slippage of the beam reinforcement through

the joints occurred earlier in the lightweight specimen, at $\mu_\delta = 2.4$ as compared to $\mu_\delta = 5.4$ for the normal weight specimen, resulting in a more pinched hysteretic behavior.

5. The assumption of a rigid joint appears to be inaccurate not only at large ductilities, but even at the yield level, under monotonic and particularly cyclic loading. The contribution of the fixed-end rotation to the total story drift under monotonic loading is about 13 percent at the yield level, increasing to 22 percent at higher ductilities. Under cyclic loading the contribution is 18 percent at the yield and increases to greater than 90 per cent at higher ductilities.

6.4 Recommended Design Improvements and Research Needs.

1. Development of new design and construction methods is needed to prevent yielding of the reinforcement at the beam-column interface, which usually triggers or accelerates total slippage of the beam reinforcement. One such method is to move the regions of the inelastic action away from the joint. This can be accomplished by: (i) bending or cutting off at a short distance from the joint some of the top and bottom beam reinforcing bars, forming a region of sufficiently lower moment capacity to be the critical one. Some research has already been conducted in this area using normal aggregate [119, 126]; or by (ii) designing haunches which sufficiently increase the moment capacity near the joint to prevent yielding of beam reinforcement at the column face. Another method consists of improving the anchorage of the reinforcement within the joint by using special mechanical devices [114] or better detailing, such as crossing the top and bottom beam reinforcement [127,143].

2. The basic causes for more rapid bond deterioration in lightweight concretes should be explored further.

3. Experiments with beam-column subassemblages having floor slabs are needed to more accurately simulate the actual conditions found at joints in buildings.

4. Analytical programs need to be developed, based on a stiffness degradation model, which include fixed-end rotations at the joint in order to study the affect of the observed deterioration on the response of framed structures to earthquake ground motions.

7. SEISMIC BEHAVIOR OF PRESTRESSED AND PRECAST R/C LINEAR ELEMENTS AND THEIR CONNECTIONS

7.1 General

7.1.1 Prestressed Concrete. In 1972, the author reviewed the state-of-the-art in prestressed and partially prestressed concrete structures and their elements [16]. He reported the conclusions reached by Blakely and Park in their historical review of the seismic resistance of prestressed concrete (1971) [144], as well as the conclusions of their tests on four full-size, precase prestressed concrete beam-column assemblies. A brief summary of these conclusions follows:

From the 1971 review:

(1) Most structures containing prestressed concrete elements which have been subjected to earthquakes have performed well. Failures which have occurred appear to have been due mainly to failure of the supporting structures or of the joint connections. However, there is very little information on the behavior of fully framed prestressed concrete structures under strong earthquakes.

(2) Although the energy absorbed by a prestressed concrete member could be the same or even larger than a similar reinforced concrete member the greater elastic recovery of the prestressed concrete member will result in a lower energy dissipation for cyclic loading. This lower energy is a drawback in seismic design. However, little is known of the energy-dissipation capacity of prestressed concrete members under high-intensity cyclic loading.

(3) High intensity cyclic loading tests of prestressed concrete members and subassemblages including different joint details is needed.

From the test results:

(1) Energy dissipation is relatively small prior to commencement of crushing in the concrete, but substantial once crushing has occurred. (2) Large post-elastic deformation can be available in prestressed concrete members, even where the transverse reinforcement satisfied only normal prestressed-concrete code requirements for shear. (3) Substantial stiffness degradation is apparent for prestressed concrete members after high-intensity cyclic loading. (4) Mortar joints between precast post-tensioned frame members can behave satisfactorily under high-intensity load reversal. (5) Prestressed-concrete framed structures can be capable of resisting moderate earthquakes without structure damage, and of withstanding severe earthquakes although structural damage may occur, with a consequent difficulty of repair back to fully prestressed condition.

In the concluding remarks of Ref. 16, the author enumerated a series of problem areas in which research was needed to improve understanding of the behavior of concrete structures under generalized excitations. The author then stated, "All the above required research applies as well to reinforced concrete as to prestressed concrete. However, in prestressed concrete other problems such as questions of the optimum degree of partial prestressing, of bonded versus unbonded prestressing tendons, the behavior of prestressed anchorage under dynamic loading, etc. still remain to be answered." The author would like to emphasize that the basic problems encountered in the seismic behavior of ordinary reinforced concrete are also present in prestressed concrete, since prestressed concrete is just a special case of reinforced concrete structure in which an initial, desirable state of compression is introduced to the concrete. The only difference is the degree of severity of these problems. (For example, one cannot expect good seismic hysteretic behavior of prestressed elements whose critical regions have not been properly confined with lateral reinforcement. These points - that the basic problems of ordinary and prestressed concrete are the same, and that the severity of the problems may differ - should be kept in mind in juding results from experiments of prestressed concrete elements.

In order to obtain a good sense of the state-of-the-art and the state-of-the-practice, up to 1977, of seismic behavior of prestressed concrete framed structures and their elements, one can review papers presented at the ERCBC Workshop held at Berkeley in 1977 [17]. Particularly appropriate are the papers by Lin and associates [145]; Park [146]; Hawkins [147]; and Park and Thompson [148]. Hawkins, in Ref. 149, has reviewed and synthesized the information presented in this workshop and several other researchers and practicing engineers have discussed it. From this review it is apparent that although the advances in knowledge about seismic behavior of prestressed concrete elements have been not as great as the advances for ordinarily reinforced concrete elements, there is sufficient evidence to formulate comprehensive seismic design recommendations for prestressed concrete [150,151]. A brief summary of some of the new information on seismic behavior of prestressed concrete elements is presented later in this section.

It is generally agreed that the response of a prestressed concrete structure to a given earthquake will be greater than that of a comparable reinforced concrete structure, because of its lower energy dissipation and viscous damping properties. However, because the use of higher concrete strength results in a smaller neutral axis depth, prestressed concrete members may sustain greater curvatures before crushing than comparable reinforced concrete members of the same flexural strength and section size. Alternatively, prestressed concrete members may be of smaller section, and therefore less mass. These factors may well counteract the effect of the smaller energy dissipation capacity under cyclic loading [152]. From the review of all the available information, it becomes apparent that proper use of prestressing can be an asset to seismic resistant construction of concrete frame structures.

7.1.2 Precast Elements. In the zones of high seismic risk in the United States, precast concrete framing is not widely used as a primary lateral load resisting system: little information exists regarding seismic behavior of this type of concrete construction. Hawkins, in Refs. 147 and 149, reviews the state-of-the-art in seismic resistance of precase concrete structures, although most of the review is devoted to precast panel construction rather than to precast concrete frames. Ikeda and associates, in Ref. 153, have reviewed the state-of-the-art of precast concrete techniques in Japan, pointing out that the main problem is the prediction of strength and deformation capacities of beam-column connections. It is clear that there is nothing wrong with the elements. The problem is in the joints between these elements. It is believed that proper use of prestressing can improve the performance of joints between precast elements. There is a tremendous potential for the use of lightweight aggregate concrete, precast, prestressed elements in seismic resistant construction.

7.2 Seismic Behavior of Prestressed Concrete Beams, Columns, and their Subassemblage

7.2.1 Beams. As with ordinarily reinforced concrete structures, it is convenient to classify prestressed beams according to stresses controlling behavior of their critical regions: i.e., Flexural and Flexural with High Shear.

7.2.1.1 Flexural Critical Regions. Hawkins [147], after analyzing the experimental results obtained in numerous experiments as well as the performance of prestressed concrete beams in real earthquakes, drew a series of conclusions. The most important conclusions are summarized below, together with some conclusions from recent studies carried out in New Zealand [148,152].

(1) Most prestressed concrete beams, when designed for loading reversals, perform well in earthquakes. Generally, deformed bar reinforcement and confinement by stirrups are necessary to provide adequate strength under moment reversals. The failures that have occurred have been due mainly to failures of the supporting structures or connections. Major consideration must be given to the strength of connections and supporting structures.

(2) Experimental flexural strengths of the beams are usually greater than theoretical flexural strengths because experimental moments reach their maximum at an extreme concrete fibre strain greater than 0.003. This is due to the extra confinement given to the beam concrete by its reinforcement and the adjacent column concrete. With stirrups and compression reinforcement, ultimate strength can increase by as much as 16 percent.

(3) Unless the first damaging load exceeds about 80 percent of the collapsed load, the capacity in the reverse direction is unaffected. If the concrete is not confined, cycling to strains greater than 0.002 induces a loss in strength and stiffness due to spalling of the compressed concrete and penetration of crushing into the core of the member. That degradation can be slowed and the ductility and energy absorption increased by the addition of either bonded compression reinforcement or confinement - preferably both. Unless confinement is provided there is a marked degradation in the flexural capacity for beams reversed cyclically and loaded to an excess of 90 percent of their flexural capacities. Confinement should be achieved by closed stirrups with a spacing not exceeding d/4.

(4) High seismic loading rates can result in strength increases of four to seven percent and ductility increases of 10 to 15 percent. It is generally appropriate for design computations to be based on static loading strengths only.

(5) Prior to crushing of the concrete or marked inelastic flow of the prestressing steel, loading-unloading curves are bilinear with ranges corresponding to crack open and crack closed conditions. The loading and unloading curves closely parallel each other with small amounts of dissipation of energy.

(6) Prestressed beams show marked elastic recoveries even after considerable inelastic deformations, leading to pinching of the hysteretic loops. Figure 29 compares beam moment-end deflection relationships for three beam-column specimens with similar theoretical flexural strengths and with prestress levels of 1160, 386 and 0 psi, (8.1, 2.7 and 0 MPa,) respectively. Energy dissipation for prestressed concrete elements is less than that for

reinforced concrete elements because of elastic recovery effects. In general, the residual tensile force in the prestressing steel is adequate to close previously open cracks. Thus, significant energy dissipation does not develop until the deformed bar reinforcement yields, the prestressing steel yields, or the concrete crushes. Recent test results of beams where flexural behavior controls inelastic response have been promising. The use of these beams in seismic resistant prestressed concrete frames should be investigated further. Most previous tests have primarily involved symmetrical arrangements of prestressed and nonprestressed steel: these tests need to be extended to other arrangements. Further study is also needed of the spacing of stirrup ties that are required to prevent buckling of nonprestressed steel under reversed loading.

7.2.1.2 Flexural Critical Regions with High Shear. As noted by Hawkins [147], there is little information available on the behavior of these prestressed critical regions. In the tests carried out by Park and his associates [148, 152], the nominal unit shear stressed developed were very small: less than $2\sqrt{f_c'}$ (psi) [$0.16\sqrt{f_c'}$ (MPa)] and less than 1/3 of the theoretical computed shear strength using ACI 318-71 [35]. Therefore, no adverse shear effects were observed. There is an urgent need for systematic studies on the behavior of the prestressed elements subjected to high shear stresses.

There is general agreement that beams should be proportioned and detailed so that they will not fail in shear. The FIP Commission [150] recommends that in calculating the design shear force the plastic hinge moments should be determined considering the possible overstrengths of the material. These enhanced plastic hinge moments may be estimated as 1.15 times the flexural capacities based on the characteristic strengths of the materials. The proposed provisions for the New Zealand Code [14] contains specific requirements for designing against shear force, neglecting the concrete's contribution in resisting shear when the design axial compressive force produces an average stress smaller than 0.1 f_c'.

7.2.1.3 Bond, Grouting and Anchorage. According to Lin and associates [145], seismic safety can be equally obtained by either bonded or unbounded construction. However, this is a controversial issue, on which the FIP Commission on Seismic Structures has prepared a special report [145]. Present FIP guidelines [150] recommend grouting the prestressing ducts in flexural members of a ductile structural frame. The New Zealand Code has similar requirements, except for special cases where post-tensional tendons may be ungrouted. Bond transfer lengths and performance under cyclic loading are very sensitive to surface conditons and to the method of release for the strand.

Careful consideration must be given to the location of tendon anchorages. They should not be placed in regions of high bending or rotation, which can adversely affect their capacity. Consideration must also be given to the flow of forces from the anchorage.

7.2.2 Columns. Except for experiments carried out by Hisada and associates [142] there has not been much research done on prestressed columns alone. Usually, columns have been studied as part of a subassemblage, in which they were stronger than the beams and hence were not critical elements. An exception to this was the joint core regions which will be discussed later. The ductility of prestressed concrete columns have been studied by Blakeley [154]. As expected, the available curvature ductility of a prestressed concrete column decreases with increased axial load level. Special transverse confining steel is necessary in prestressed columns (as it is for reinforced concrete columns) once the axial load exceeds some nominal value such as 0.1 P_0 where: P_0 = strength of columns when load is applied with zero eccentricity.

Prestressing can improve the behavior of reinforced concrete columns [142] and therefore of the whole frame, provided the peculiarities of prestressing are considered in the design as well as in the detailing of the columns. Figure 30 illustrates an example of post-tension prestressing the outer columns of the first 5 stories of an 18 story building (to reduce the possibility of tensile cracking strength during severe earthquakes). This application has been discussed by Ohmori [84], Muto [94] and Hisada and associates [142].

There is a need for experimental work on partially prestressed columns under severe seismic actions. Among the parameters that need to be studied the following deserves special attention:

• optimum degree of prestressing, and optimum location of the pressure line;

• quantity of confining steel necessary to achieve adequate rotation ductility, particularly under high compressive loads, and to prevent buckling of the bars;

• the affects of unbonded tendons, particularly when used continuously over several column stories.

7.2.3 <u>Beam-Column Joints</u>: Following design criteria similar to that used for ordinarily reinforced concrete structures, the FIP Commission on seismic structures [150] recommends: "The connections between members in prestressed concrete construction should be carefully designed for effectiveness at all earthquake limit states, on the following basis:

(a) Connections should be checked for both seismic stresses and deformations.

(b) The load-carrying capacities of connections should not be less than those of the adjacent structural members.

(c) Connections should be capable of failing in a ductile manner. "

In their commentary the FIP Commission emphasizes that inelastic loading cycles (particularly those involving not only load but also deformation reversals) can result in a degradation of the concrete shear-resisting mechanism due to breakdown of the joint core, caused by alternating bond force and diagonal tension cracking.

The above design philosophy is clear and well accepted. However, adequate provisions, methods and rules for quantifying and practically applying this philosophy are still lacking despite improvements in understanding hysteretic behavior of beam-column joints. Most of the studies have been related to the strength of the joint, very little has been done regarding prediction of stiffness and its degradation with increasingly severe cyclic loading, or with the prediction of deformation capacity and energy dissipation capacities.

The work of Park and his associates [146,152] has significantly increased knowledge of the effects of prestressing on joint behavior. Their work showed that serious difficulty in preventing joint core distress during severe seismic loading can only be minimized by careful proportioning and detailing. Their main findings follow:

(1) The ACI 318-71 Appendix A [36] approach for joint core shear strength cannot be regarded as adequate for plane frames subjected to intense cycles of seismic loading. It fails to make any provision for vertical shear reinforcement in the plane of bending.

(2) The use of a reasonable level of prestress through a central tendon improved the hysteretic behavior of the joints.

(3) The contribution of the concrete to shear strength should be neglected except when the mean column compressive stress exceeds to 0.1 f_c'.

(4) The inclusion of vertical shear reinforcement within beam-column joint cores, in the form on intermediate column bars, and horizontal shear reinforcement, in the form of ties, allows the joint core shear force to be resisted more effectively than when intermediate column bars are not present (Fig. 31).

(5) The draft of the New Zealand Concrete Design Code [14] recommends the provision of the vertical shear reinforcement to transmit vertical shear forces within the joint core. The amount of horizontal and vertical shear reinforcement required by this draft Code approach was found to be safe but rather conservative.

Although the above results led to improved understanding of the hysteretic behavior of prestressed concrete beam column joints, research is needed in the following areas:

(1) The actual contribution of concrete to joint strength, stiffness and energy dissipation capacity when subjected to different levels of compressive stress.

(2) Other means of vertical joint shear reinforcement.

(3) Maximum bar diameters allowable for longitudinal steel to prevent total slippage through the joint core.

(4) The affect of unbounded tendons.

(5) The potentials of moving the critical regions away from the face of the columns.

7.3 Seismic Behavior Precast Concrete Beam, Column and their Connections

As discussed in Section 7.1.2, the main problem in using these elements is associated with their connections. As noted by Hawkins [147], while many types of connections have been developed [155,156] more information is needed regarding the behavior of these connections under severe earthquake loading conditions. A comprehensive experimental research program is needed where these connections, as well as those already in use, will be studied under simulated seismic conditions. Meanwhile, it is recommended that designers and fabricators of these precast elements try to locate the connections so they can be easily constructed and are not subjected to severe simultaneous bending, shear and axial forces. An example of proper location of field connections is shown in Fig. 32.

7.4 Concluding Remarks

Prestressed and precast linear concrete elements are not widely used to form primary seismic resistant structural systems. The amount of research in this area has been relatively small compared with that on ordinarily reinforced concrete, and some fundamental questions remain unanswered. Nonetheless, in the last decade, there have been significant advances in understanding problems introduced by these techniques of reinforced concrete construction.

There is tremendous potential in the use of prestressed and precast lightweight concrete structural elements. To realize this potential quickly, it is necessary to recognize - that prestressed and precast concrete elements are just a particular case of R/C structures and practically all drawbacks of ordinary R/C elements are also present in prestressed and precast elements. Therefore, existing knowledge of seismic behavior of the ordinary R/C elements should be used. The problems to concentrate on are those that are peculiar to prestressing (i.e. problems of anchorage, bond, transfer, grouting, type of steel and level of prestressing); and to the precasting technique (like the problems of joint).

8. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE RESEARCH AND DEVELOPMENTS

8.1 Summary

Significant advances have been made in the last ten years in understanding seismic behavior of structural concrete linear elements and their connections. This improved understanding has had some impact in earthquake resistant design of R/C structures where these elements are used. However, much of present knowledge has not yet been practically applied. There are several problems in predicting seismic behavior of these linear elements and their connections. Some of these problems are of a general nature and apply to all types of elements, regardless of the material used (e.g. problems in predicting demand due to uncertainties about the ground motion and the overall response of the structure). There are other problems, inherent to the type of member and associated with the peculiar sensitivity of reinforced concrete construction to all those aspects which affect structural behavior - design, construction, maintenance, modification, and repair - which should be considered in order to obtain efficient seismic resistant construction.

Problems of a general nature have been discussed in section two. The seismic behavior of any element of a structure depends upon the interaction of the ground motion and the structure; there are many uncertainties in predicting both ground motion and structural response. All these uncertainties must be considered in order to judge the reliability of experimental results and to assess the implications of these results for design and construction of seismic resistant structures. To characterize these uncertainties properly, data from field and laboratory studies must be collected and statistically reviewed. Then studies may be carried out on the probability of failure of R/C elements.

Section two emphasizes the importance of loading history in the behavior of elements. The importance of properly selecting a structural layout and choosing the material to be used is also discussed.

The requirements for suitable seismic resistant structural materials are discussed. The relatively low value of strength per unit weight of normal weight concrete suggests the desirability of using lightweight concrete. The use of precast, partially prestressed lightweight aggregate concrete elements has tremendous potential in seismic resistant construction. However, the technology of lightweight aggregate; the problems of determining the optimum degree of prestressing; and the problems of connections of prefabricated elements, have not yet been resolved. Thus the most suitable R/C material for earthquake resistant design is still cast-in-place, ordinarily reinforced, normal weight concrete.

Section two also discusses the importance of studying the seismic behavior of basic structural components and their subassemblages, rather than the response of a whole structure.

A review of the inherent problems of linear reinforced concrete members and their connections shows that no general theory has been formulated to accurately predict the real seismic behavior (stiffness, strength, deformation, and energy dissipation capacities and their variation with load) of such structural components. It is doubtful that such a theory will ever be developed. However, there have been significant accomplishments in the understanding of such behavior, particularly of R/C elements that are used in plane moment-resisting frames subjected to unidirectional (1D) loading conditions in the plane. For these elements not only have the problems been determined, but the different sources of the problems have also been identified. The author considers these advances of paramount importance and would like to emphasize the need to present these advances to the profession. The author considers this to be more important than developing simple empirical rules for the design of standard elements. If the designer knows what the problems and their sources are, he has two possibilities for coping with them. First, he can try to avoid them. Since he knows the sources of the problems, if he cannot avoid them, he can try to minimize them by proper design, particularly proportioning and detailing. Two typical examples follow:

Most failures of R/C linear elements are caused by the development of high shear in flexural critical regions. The designer can avoid such problems by proper selection of structural form, selecting relatively slender members and/or using a low percentage of steel reinforcement of low yielding strength and strain hardening characteristics. Since such failures are due to sliding shear, designers can avoid or sufficiently delay the failure of such members by proper use and detailing of special web reinforcement in the flexural critical regions.

Another problem that has been observed in seismic behavior is the degradation in stiffness and strength of beam-column subassemblages with repeated cycles of deformation reversals. This problem occurs at the beam-column joints; its sources have been identified as high shear and/or high bond stress through the joint. The designer can avoid this problem by selecting wider columns, and beams with a low percentage of steel with low yielding strength and strain hardening characteristics. Or he can avoid the formation of beam plastic regions at the faces of the columns. If this cannot be done, proper detailing of the reinforcement of the beam, column, and joint can minimize the detrimental consequences of stiffness and strength degradation.

Following, with the presentation of the conclusions, there is a summary of advances in the design and understanding of seismic behavior of normal weight R/C elements and their cast-in-place subassemblages under 1D loading conditions. There has been very little research for 2D or 3D loading. However, some parameters influencing the seismic behavior of frame subassemblages under two dimensional-lateral motions have been identified.

There have been some significant advances in understanding behavior of lightweight concrete. Some of the peculiarities of this type of concrete have been identified by comparing its behavior with the behavior of similar normal weight concrete. These peculiarities include: a lower gain in strength and ductility with confinement (particularly with high strength [e.g. greater than 4,000 psi]); lower bond; and lower shear transfer. More comprehensive studies are needed of the mechanical characteristics of this type of concrete and its interaction with reinforcing steel under seismic conditions.

The amount of research in the area of prestressed and precast linear concrete elements has been small. However, there have been some advances in the proper use of prestressing, particularly for improving behavior of beam-column joints and columns in tall buildings. The main problem for precast construction is connection. Although many types of connections have been suggested, and some used, there is no available information about their behavior under seismic loading.

8.2 Conclusions

The following conclusions emphasize findings which have helped to advance the design and construction of normal weight R/C elements and their cast-in-place subassemblages subjected to 1D loading conditions. General observations applying to all members are presented first. Then observations for beams, columns and their connections are presented separately.

Reliable methods are lacking to predict demands, particularly deformation and energy dissipation demands, that can be expected during a structure's response to extreme earthquake shaking. Therefore, it is highly desirable to design R/C elements and their subassemblages so that they will be capable of dissipating the largest possible amount of energy through stable hysteretic behavior. Special attention should be paid to proportioning and detailing. The following recommendations are designed to achieve such stable, tough behavior.

8.2.1 Beams. Most of the following observations apply to the design of the potential beam critical regions.

(1) It is essential to provide sufficient shear capacity in potential critical (plastic hinge) regions to develop the required flexural deformation and energy dissipation capacities.

(2) Lower tension steel contents, ρ, are recommended than those presently allowed by R/C codes.

(3) It is recommended that beams be designed so that, at their connection with columns, they have a larger positive moment capacity than presently required by seismic codes ($\rho'/\rho \geq 0.75$ has been recommended).

(4) The location of splicing of main reinforcing bars should be carefully established. As much as economically feasible, curtailing of the main bars should be avoided.

(5) The effectiveness of different arrangements of transverse steel in confining concrete has been studied and constitutive laws for such confined concrete have been formulated.

(6) Present seismic code requirements for beam confinement are not adequate when large ductility is demanded.

(7) To prevent premature buckling of main reinforcing bars, each of these bars should be supported laterally by a corner of a tie and tie spacing should not exceed six bar diameters.

(8) The use of beams where the nominal unit shear stress, v_{max}, can exceed $6\sqrt{f'_c\ (psi)}$ $(0.5\sqrt{f'_c\ (MPa)})$ should be avoided.

(9) Present code requirements result in satisfactory hysteretic behavior when v_{max} is $\leq 3\sqrt{f'_c\ (psi)}$ $(0.25\sqrt{f'_c\ (MPa)})$.

(10) When v_{max} is in the range of $3\sqrt{f'_c\ (psi)}$ to $6\sqrt{f'_c\ (psi)}$ $((0.25\sqrt{f'_c\ (MPa)}$ to $0.5\sqrt{f'_c\ (MPa)})$, it is necessary to use special web reinforcement. Although the use of intermediate longitudinal bars improves hysteretic behavior, the addition of diagonal reinforcement seems to be more effective in controlling sliding shear at critical regions.

(11) Conventional seismic resistant design is inadequate for coupling beams, of coupled shear wall systems, which have V_ud/M_u ratios of one or less. The energy dissipation capacity (ductility and useful stable strength) can be improved by placing the main reinforcement diagonally in the beams.

8.2.2 Columns. These elements are still the most susceptible to failure in destructive earthquakes, particularly when subjected to high axial and shear forces. This is because of the sensitivity of shear stress to variations in the values of many of the factors affecting such column stress.

(1) Short columns designed and constructed according to present U.S. seismic codes can dissipate moderate amounts of energy through inelastic deformations. This can be adequate for ductile moment-resisting frames which are properly designed, constructed, and maintained and in which the short columns are not subjected to significant fluctuations of axial force.

(2) In the case of large flexural ductility demands, the contribution of concrete to shear resistance should be ignored.

(3) Circular spiral is the most effective transverse reinforcement to confine concrete and prevent the main reinforcing bars from buckling.

(4) New types of column reinforcement have been developed in Japan. A combination of spiral and square hoops resulted in excellent hysteretic behavior.

(5) Because joint core behavior can lead to some damage of the concrete cover of the column, the column strength computation should be based on the strength of the core area only.

8.2.3 Beam-Column Joints. Design criteria have been forumlated for this type of joint. The criteria for the strength of the joint is that the beam-column joint should be the strongest and stiffest component of a basic moment-resisting frame subassemblage. While this usually has been so in the past, it might not be so in future structures, because while more stringent requirements for seismic design of beams and columns have recently been included in codes, no changes have been introduced for the design of joints. Research results have indicated that:

(1) The effectiveness of concrete to resist shear should only be considered when there is a compressive load on the column which exceeds $0.1f'_c A_g$.

(2) Vertical shear reinforcement should be provided to help transfer vertical shear force to complete the truss mechanism at the joint core. Vertical column bars should be used around the perimeter of the column section with spacing not exceeding six in. (150 mm).

(3) For exterior beam-column joints, if plastic hinging occurs in the beam at the column face it is recommedned that the diameter of the lontitudinal column bars should not exceed 1/25th or 1/20th of the beam depth (for 55 and 40 grade steel, respectively).

(4) For interior beam column joints, if plastic hinging occurs in the beams at the column face it is recommended that the maximum diameter of the longitudinal beam reinforcing bars should not exceed 1/35th or 1/25th of the column depth (for 55 and 40 grade steel, respectively). The diameter of longitudinal column bars are limited as for exterior joints.

(5) If plastic hinging occurs in the beam at the column face, in determining the anchorage length of beam steel it is necessary to distinguish between the effectiveness of the bond offered by unconfined concrete in the column cover (which is small and should be neglected) and that offered by the confined concrete core. In exterior joints, the anchorage should be considered to begin within the joint core at a distance of either one-half the column

depth or ten bar diameters, whichever is closer to the column face where the steel enters.

(6) Performance of exterior joints can be improved by using a beam stub at the far column face where the longitudinal beam bars can be anchored.

(7) Significant bond deterioration occurs at the joint core from load reversals cyclically applied to the beam bars. This results in beam fixed-end rotations, particularly when the stress applied to the beam bars entering the columns equals or exceeds yield.

(8) To avoid detrimental beam fixed-end rotations, beam hinges adjacent to column faces should be eliminated. Practical techniques to accomplish this have been suggested, tested, and proven to be satisfactory.

8.2.4 <u>2D and 3D Loadings</u>. The following observations are of a tentative nature, because of insufficient data.

(1) 2D column displacement ductility demands about twice as large as 1D ductilities are typical at a 1D displacement ductility of about five or more.

(2) To avoid difficulties under 2D it is recommended that frames be designed so that column displacement ductility demands under 1D are restricted to two.

(3) While compressive axial loads have little influence on column behavior under 2D loading, tensile axial loads substantially reduce the stiffness and shear capacity at low loads.

(4) Theoretically, for a symmetrical two-way frame, joint design for biaxial shear leads to approximately twice the shear required for uniaxial shear design. Because this can create serious practical problems, it is suggested that beam hinges adjacent to column face be eliminated.

8.2.5 <u>Use of Lightweight Aggregate Concrete</u>. Because of the relatively meager data available, the following observations are of a preliminary nature.

(1) The effectiveness of the confinement, bond and shear transfer of lightweight aggregate concrete is inferior to that of normal weight aggregate concrete of similar strength. The higher the strength of the concrete the larger the difference in confinement effectiveness. Furthermore, lightweight aggregate concrete has higher creep. Therefore caution should be used in applying equations or seismic code provisions derived for normal weight aggregate to lightweight aggregate concrete, particularly in designing columns.

(2) Under cyclic loading, the energy dissipated by beam-column subassemblages cast of lightweight aggregate concrete is significantly smaller than that of similar normal weight concrete subassemblages.

(3) The compressive strength of lightweight aggregate concrete used in seismic resistance construction should be limited according to the mechanical characteristics of the aggregate.

8.2.6 <u>Use of Prestressed and Precast Techniques</u>. In addition to the problems common to any kind of reinforced concrete elements, the main findings of the reviewed research are:

(1) Prestressed beams show marked elastic recoveries even after considerable inelastic deformations, leading to pinching of the hysteretic loops.

(2) Energy dissipation of prestressed concrete elements can be increased, and degradation of stiffness decreased, by the proper addition of bonded compression and transverse (confinement) reinforcements.

(3) Although high seismic loading rates of prestressed elements can result in strength increases of four to seven percent, and ductility increases of 10 to 15 percent, it is recommended that design computations can be based on static loading strengths only.

(4) The use of a reasonable level of prestressing through a central tendon improves hysteretic behavior of joint.

(5) The use of prestressing can improve behavior of ordinarily reinforced concrete exterior columns in tall slender buildings by decreasing the possibility of cracking due to tensile forces originated by overturning moments.

(6) Use of prestressing can improve the behavior of connections between linear elements.

(7) The use of prestressed and precast lightweight concrete structural elements has great potential for seismic resistant construction.

8.3 Recommendations for Future Research and Developments.

Among the different recommendations formulated in this report the following deserve special mention:

(1) Perform integrated analytical and experimental research on the three dimensional behavior of actual structures under realistic seismic loading conditions to determine the demands on different structural components. In order to carry out more realistic experiments than has been done up to date it is important to determine the expected loading or deformation histories that the structural elements will undergo. Seismic performance of R/C structures is very sensitive, not only to how the structures have been designed and detailed, but also to how they are constructed, and to the modifications, maintenance, and repair which they can undergo before an earthquake strikes. All these aspects must be considered in establishing design criteria.

(2) Improve quality control of the R/C materials. Statistical data regarding mechanical characteristics of the material from existing structures should be collected and studied.

(3) Perform experiments to improve prediction of the interface shear transfer in plastic hinge regions of beams and columns subjected to generalized loadings.

(4) Perform experiments under seismic loading conditions, on the contribution of the floor slab to: development of beam flexural capacity; behavior of the beam-column joint; and overall strength, stiffness, deformation, and energy dissipation capacity of basic frame subassemblages.

(5) Perform experiments to study the behavior of columns and beam-column joints subjected to two and three-dimensional loadings. Emphasis should be

placed on the effects of high shears and the fluctuation from high compressive to high tension axial forces.

(6) Conduct statistical studies of the variation of v^R_{max} / v^C_u in elements of existing buildings.

(7) Perform experimental studies of behavior of lapped and mechanical splices under high intensity load reversals and at different loading (strain) rates.

(8) Perform experiments to study behavior of construction joints in beams and columns.

(9) Perform experimental studies to establish reliable bond-slippage constitutive law for the beam's reinforcing bars along the confined concrete of beam-column joints.

(10) Conduct analytical studies of how the fixed-end rotations at the beam ends of column faces affects seismic response of framed structures.

(11) Conduct integrated experimental and analytical studies on the seismic behavior of reinforced lightweight aggregate concrete elements, with emphasis on: the effectiveness of confinement, bond, and shear transfer of such concrete; the higher rate of creep for lightweight than for similar normal weight; and how that higher creep can effect the seismic performance of framed structures.

(12) Conduct coordinated analytical and experimental studies to define: the degree of stiffness; damping; abruptness of failure; and hysteretic behavior of prestressed concrete subassemblages. These subassemblages should contain combinations of prestressed tendons and deformed bar reinforcements similar to those likely to be found in practice.

(13) Make generic studies of hysteretic behavior of different types of connections between precast elements. These studies should cover non-tensioned and post-tensioned connections subjected to loading intensities and histories similar to those which would exist during extreme earthquakes. These studies should examine precast elements of various types (particularly lightweight prestressed) and various cross sections.

(14) Conduct research programs which examine the applicability of reduced ductility and strength requirements for areas other than those of highest seismicity.

It is hoped that this report can serve as a basis for spirited discussions at the Symposium, and that these discussions will contribute toward the solution of the many problems and questions that have been raised here. Because of the complex nature of these problems, international collaboration is needed between practitioners, educators, researchers, and representatives from industry and government agencies in the field of earthquake resistant construction.

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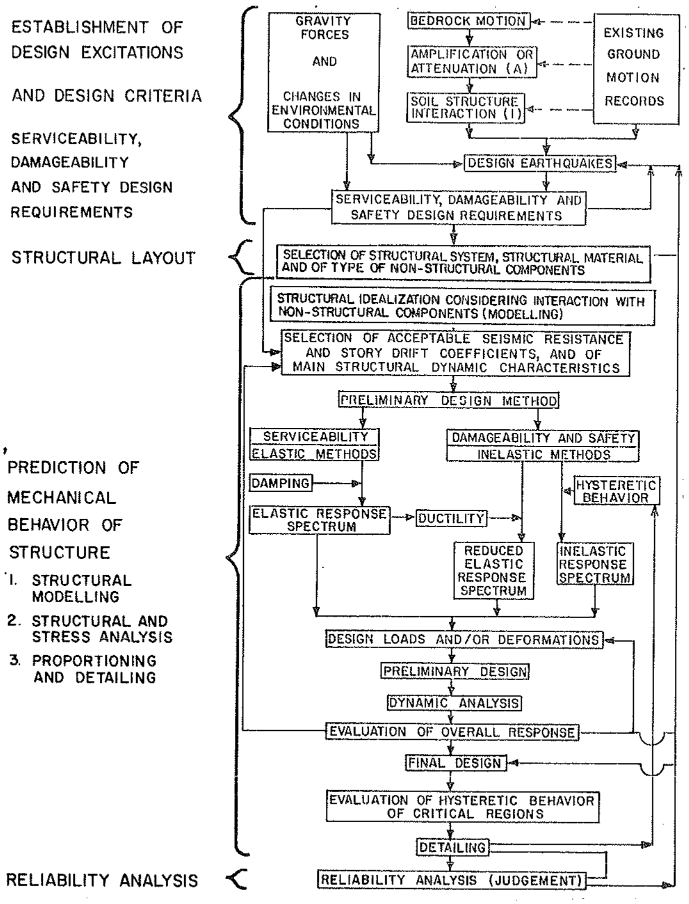
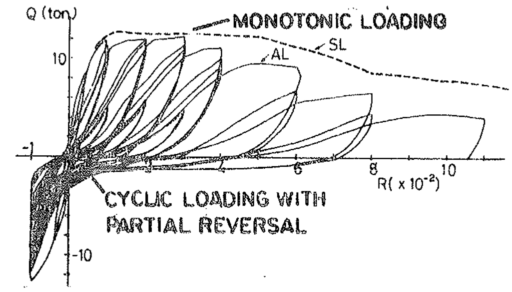
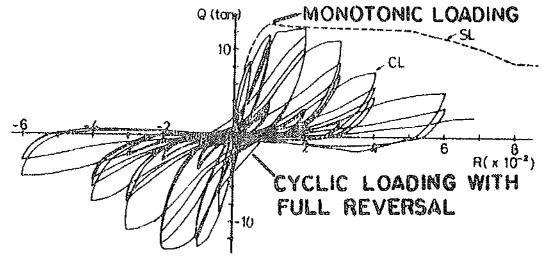


FIG.1 FLOW DIAGRAM OF GENERAL ASPECTS AND STEPS INVOLVED IN SEISMIC-RESISTANT DESIGN [21]



(a) Monotonic vs. Cyclic with Partial Reversal



(b) Monotonic vs. Cyclic with Full Reversal

FIG. 2 INFLUENCE OF LOADING HISTORY ON DEFORMATION BEHAVIOR OF COLUMNS [27]

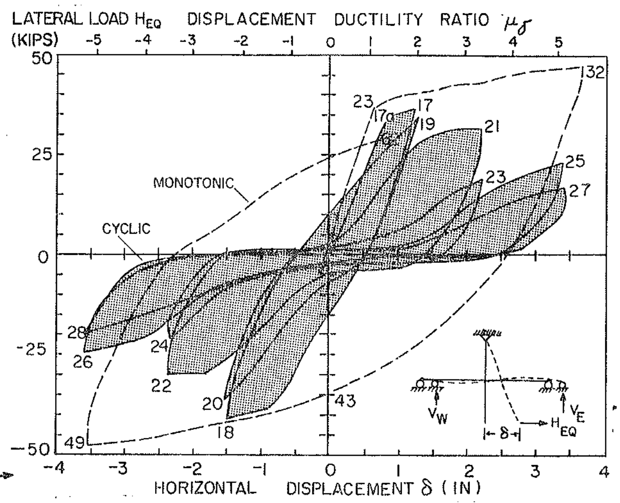


FIG.3 INFLUENCE OF LOADING HISTORY ON LATERAL LOAD-DISPLACEMENT OF BEAM-COLUMN SUBASSEMBLAGE [29]

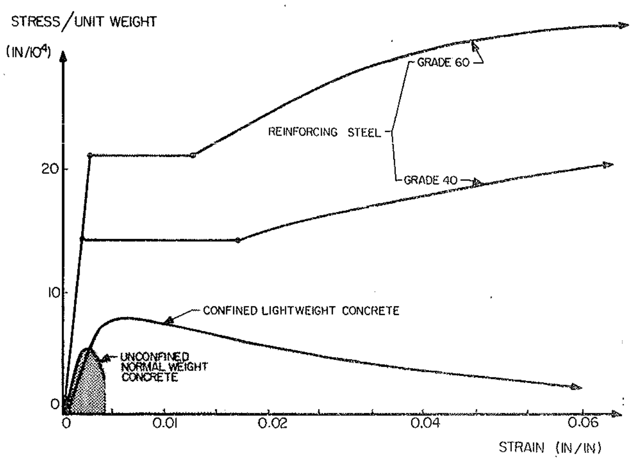
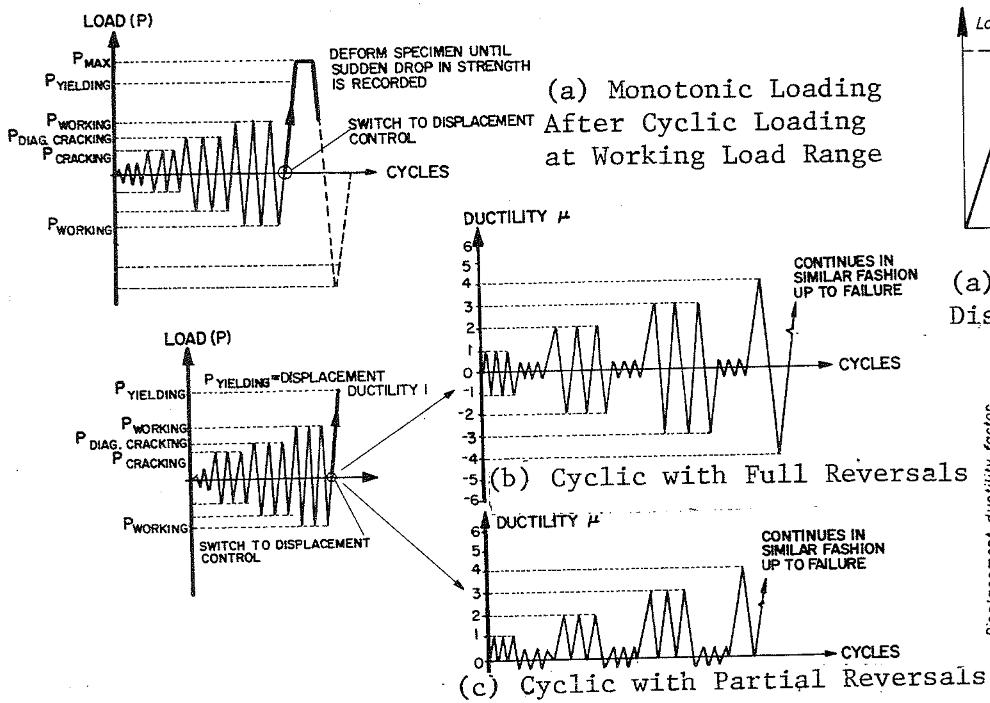


FIG.4 STRESS/UNIT WEIGHT STRAIN DIAGRAMS FOR DIFFERENT R/C STRUCTURAL MATERIALS [19]

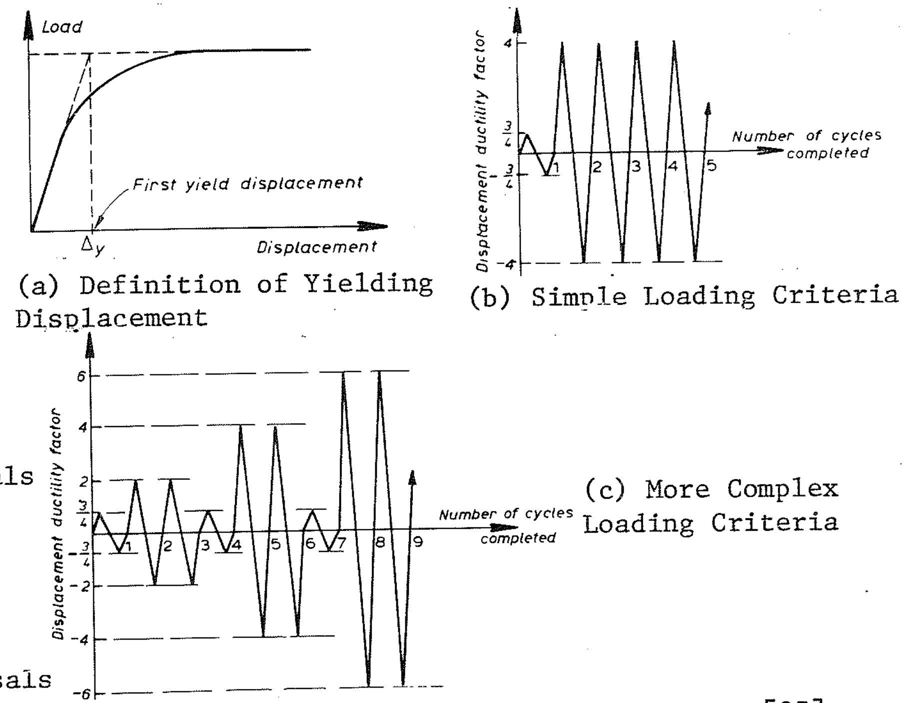


(a) Monotonic Loading After Cyclic Loading at Working Load Range

(b) Cyclic with Full Reversals

(c) Cyclic with Partial Reversals

FIG.5 LOADING HISTORIES USED IN EXPERIMENTS AT BERKELEY

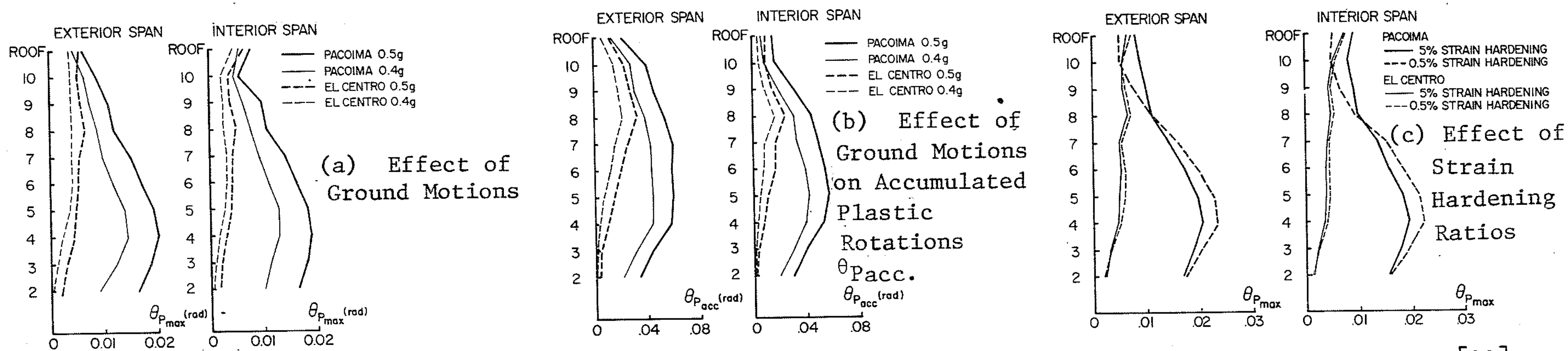


(a) Definition of Yielding Displacement

(b) Simple Loading Criteria

(c) More Complex Loading Criteria

FIG.6 LOADING CRITERIA USED IN NEW ZEALAND [25]



(a) Effect of Ground Motions

(b) Effect of Ground Motions on Accumulated Plastic Rotations $\theta_{Pacc.}$

(c) Effect of Strain Hardening Ratios

FIG. 7 BEAM PLASTIC ROTATION REQUIREMENTS FOR DIFFERENT GROUND MOTION AND STRAIN HARDENING RATIOS [33]

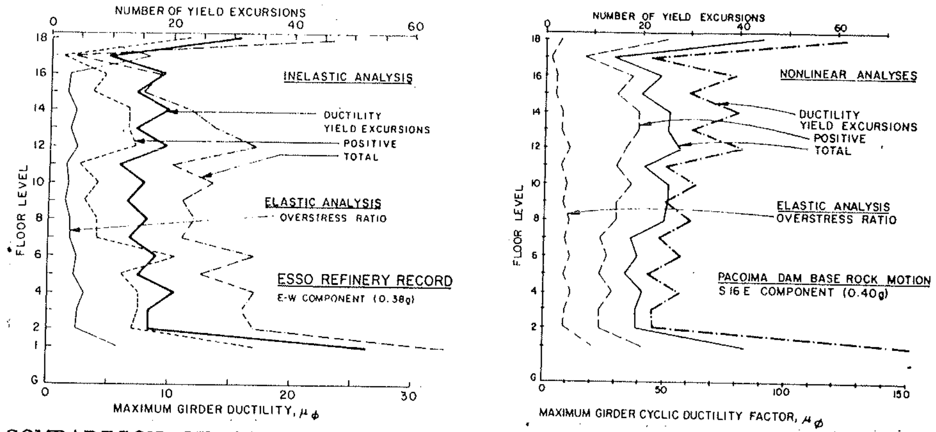


FIG. 8 COMPARISON OF COUPLING GIRDER CURVATURE DUCTILITY AND NUMBER OF YIELD EXCURSION REQUIRED DURING RESPONSE OF COUPLED WALL SYSTEMS TO DIFFERENT GROUND MOTIONS [34]

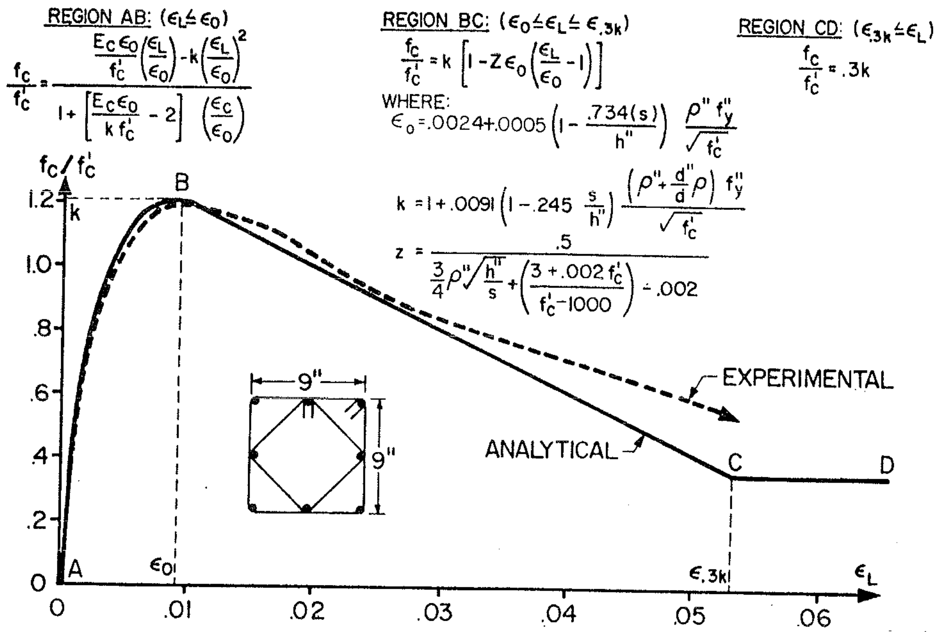


FIG.9 COMPARISON OF NEW ANALYTICAL CURVE AND EXPERIMENTAL RESULTS FOR CONFINED CONCRETE WITH LONGITUDINAL REINFORCEMENT [57]

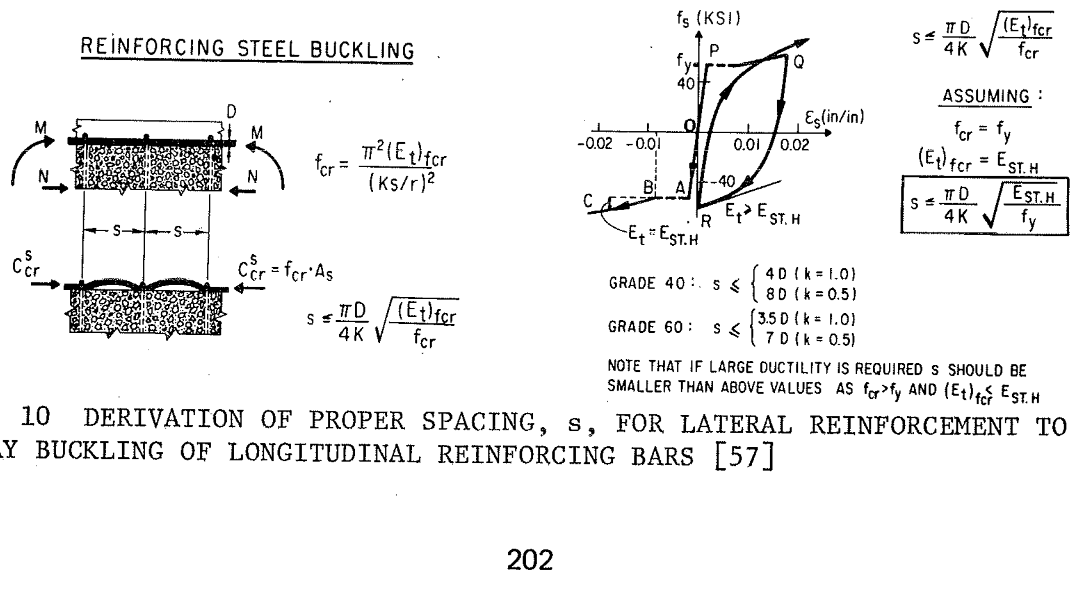
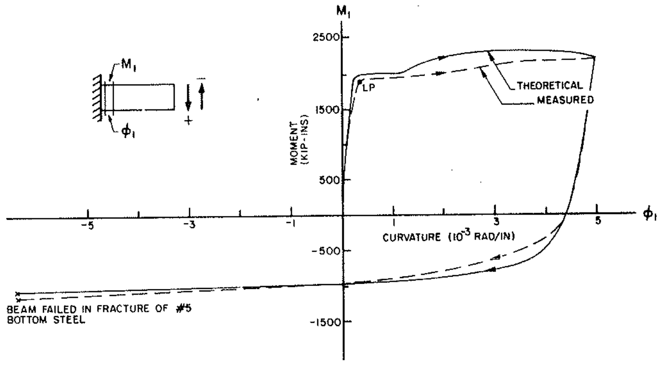
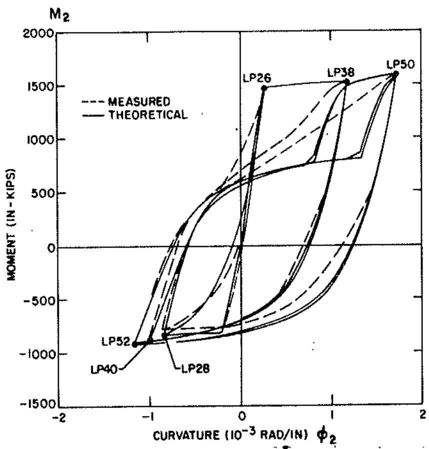


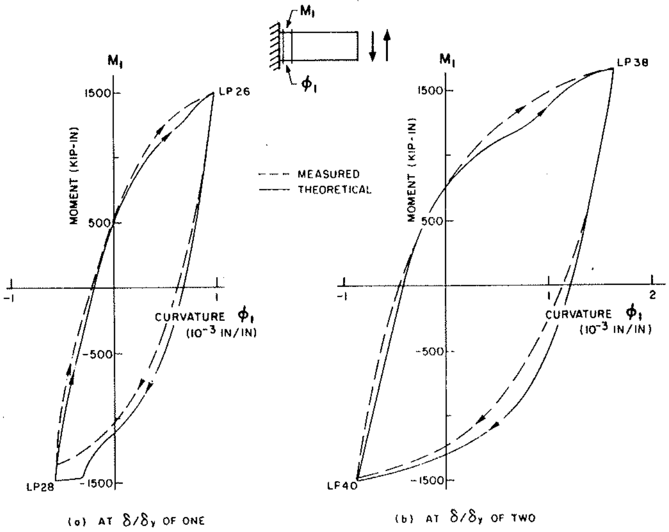
FIG. 10 DERIVATION OF PROPER SPACING, s, FOR LATERAL REINFORCEMENT TO DELAY BUCKLING OF LONGITUDINAL REINFORCING BARS [57]



(a) Monotonic Loading and Unloading

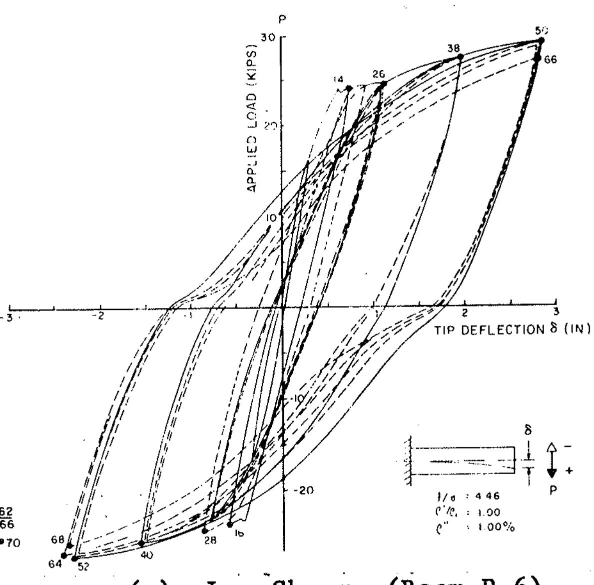


(b) Cyclic Bending Reversals (Beam R-3) ρ'/ρ_t = 0.53

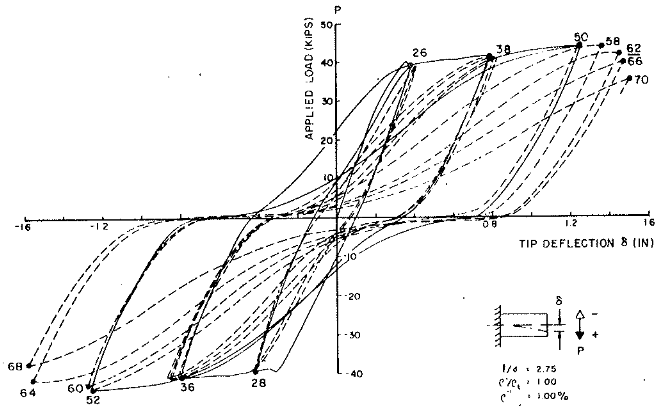


(c) Cyclic Bending Reversals (Beam R-6) ρ'/ρ_t = 1.00

FIG.11 COMPARISON OF EXPERIMENT- AND ANALYTICAL PREDICTION OF MOMENT-CURVATURE RELATIONSHIPS [69]



(a) Low Shear (Beam R-6)



(b) High Shear (Beam R-5)

FIG.12 COMPARISON OF HYSTERETIC BEHAVIOR OF FLEXURAL MEMBERS AS AFFECTED BY DIFFERENT AMOUNTS OF SHEAR FORCES [40]

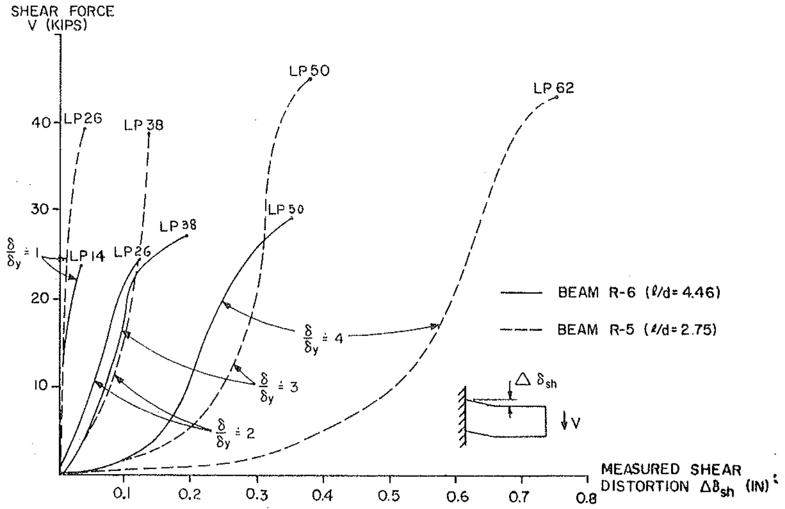
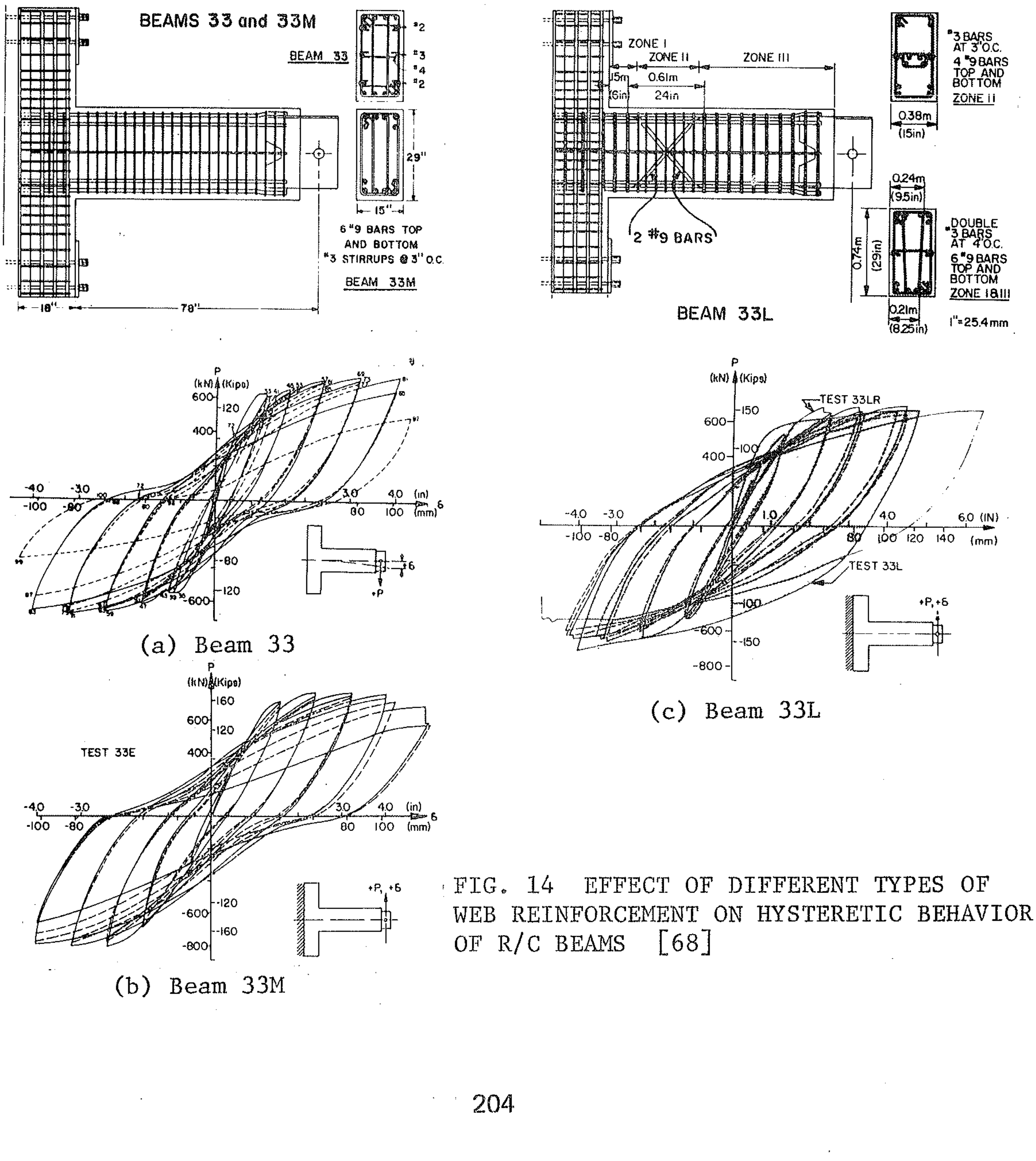


FIG. 13 COMPARISON BETWEEN SHEAR FORCE-SHEAR DISTORTION RESPONSE OF BEAMS R-5 and R-6 [40]



(a) Beam 33

(b) Beam 33M

(c) Beam 33L

FIG. 14 EFFECT OF DIFFERENT TYPES OF WEB REINFORCEMENT ON HYSTERETIC BEHAVIOR OF R/C BEAMS [68]

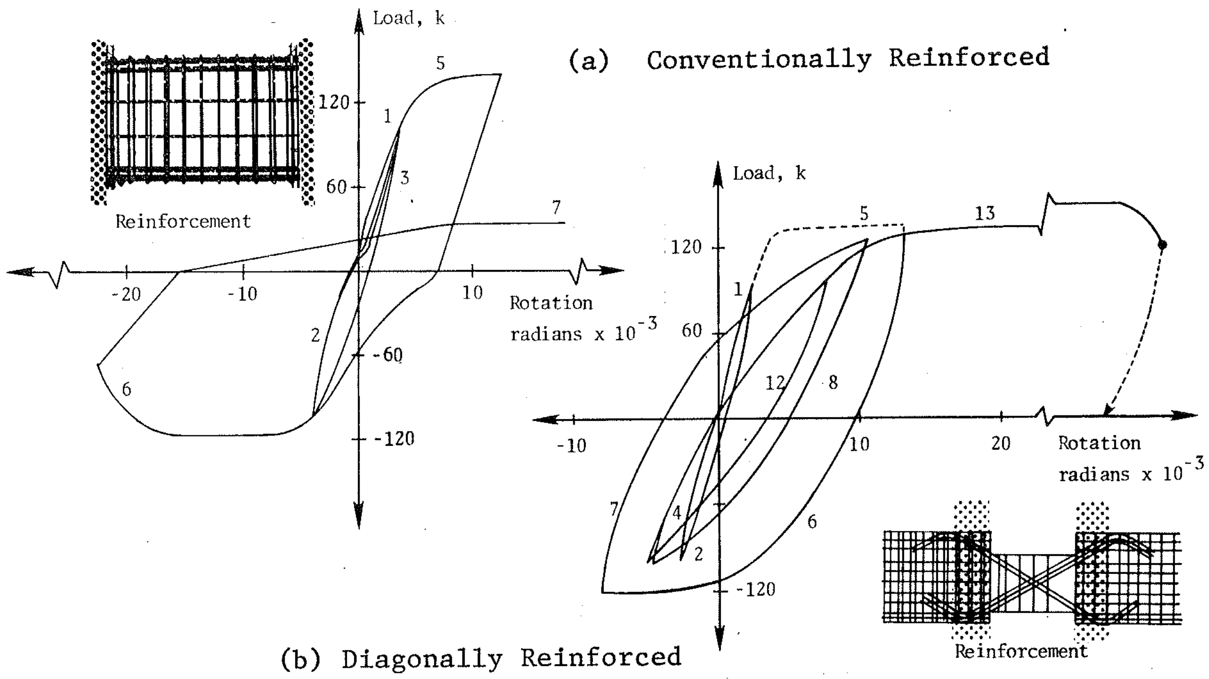
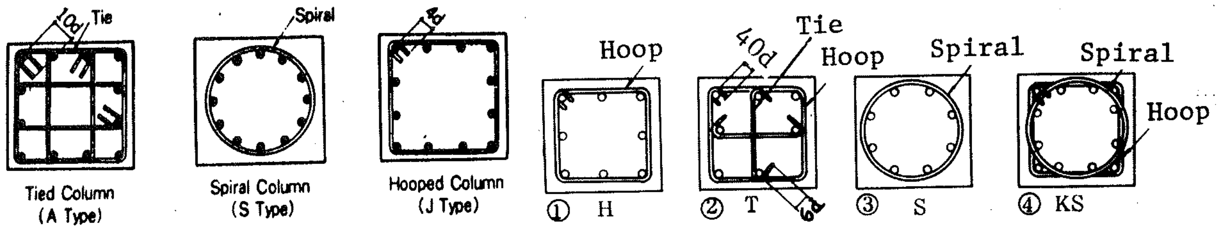
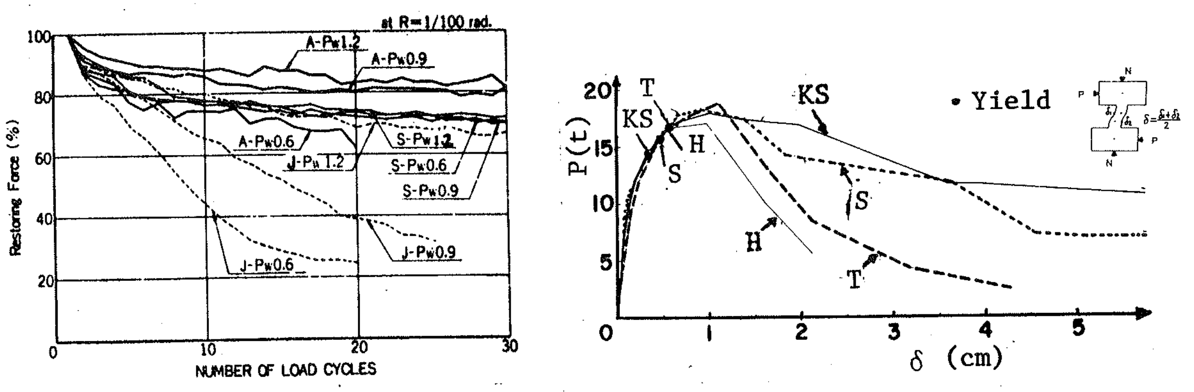


FIG. 15 LOAD-ROTATION RELATIONSHIPS FOR R/C COUPLING BEAMS [82]



(a) Typical Kinds of Lateral Reinforcement

(c) Typical and New Types of Lateral Reinforcement



(b) Reduction of Restoring Force of Columns in Fig. 10(a), with Increased Number of Cycles

(d) Envelope of Hysteretic Behavior of Columns in Fig. 16(c)

FIG. 16 EFFECT OF DIFFERENT TYPES OF LATERAL REINFORCEMENT ON R/C COLUMN BEHAVIOR UNDER CYCLIC DEFORMATIONS [84, 141, 142]

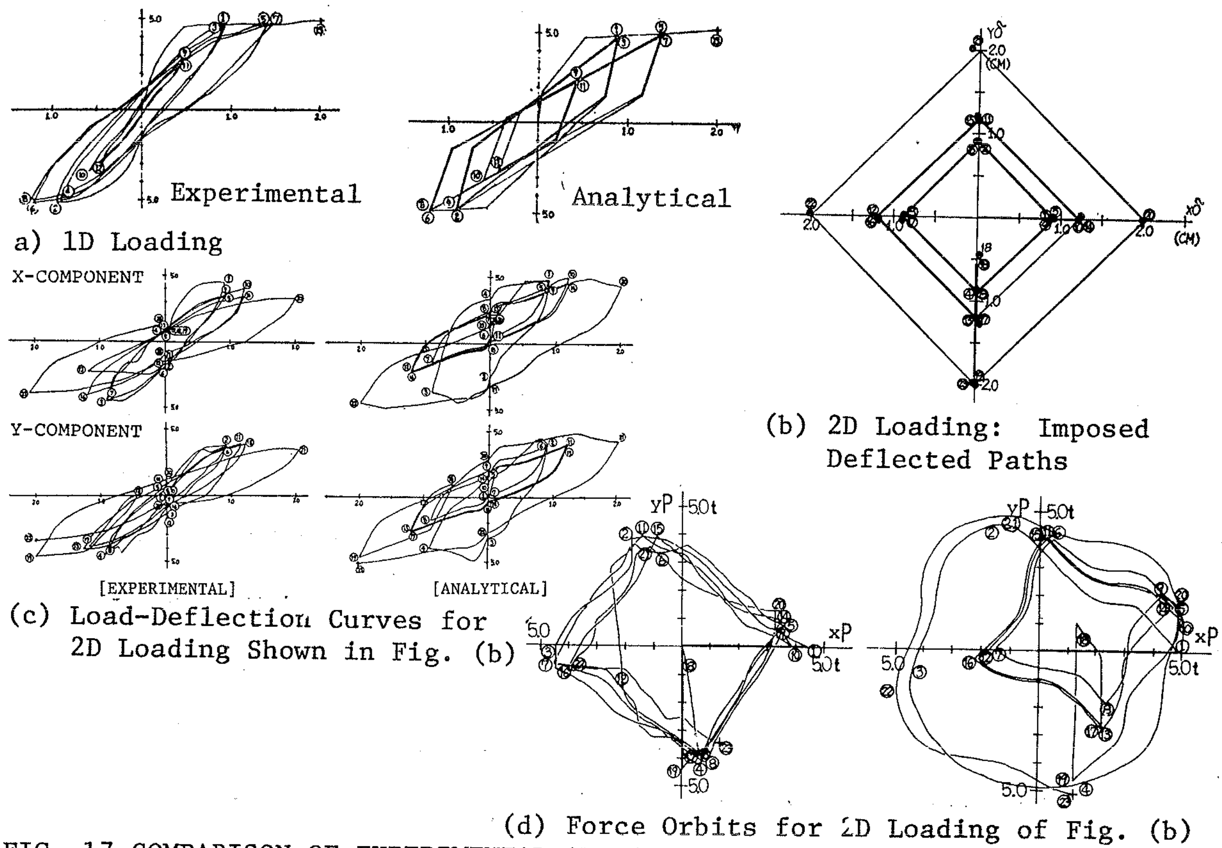
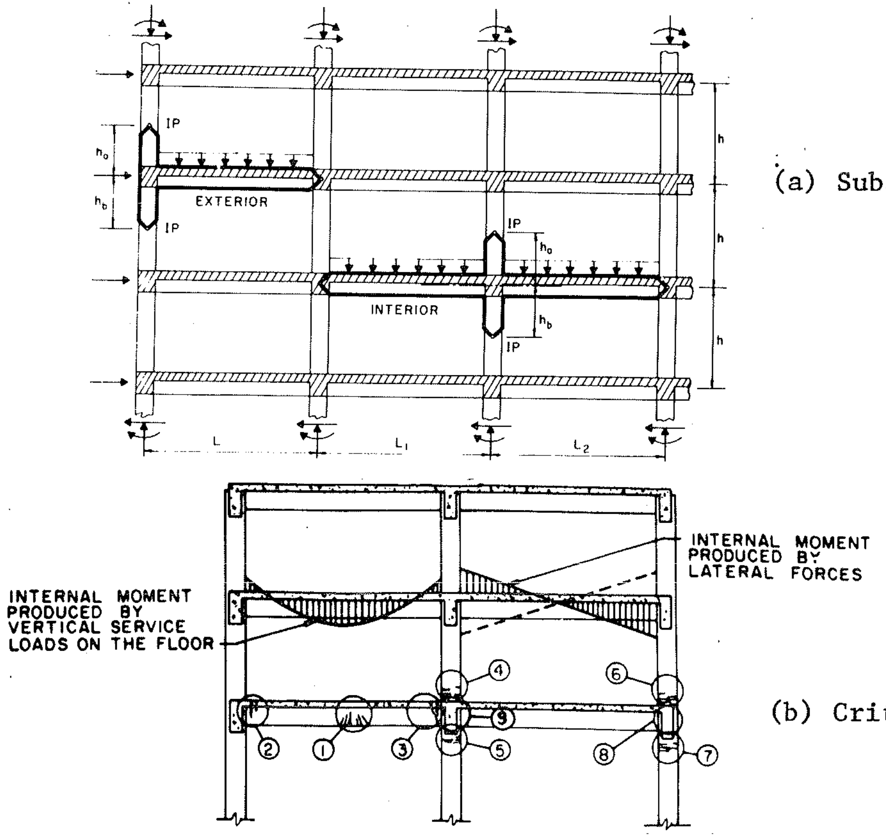


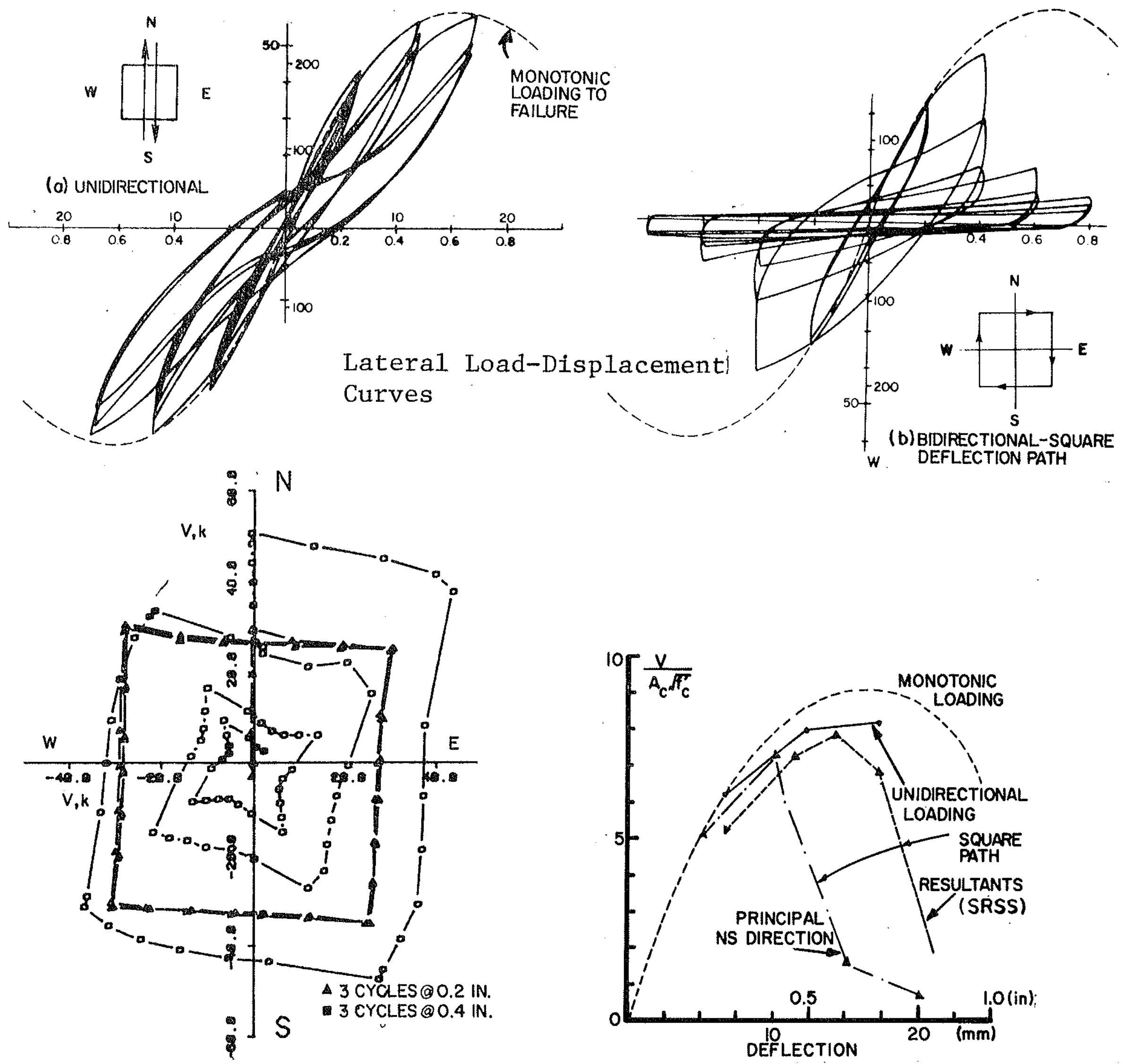
FIG. 17 COMPARISON OF EXPERIMENTAL AND ANALYTICAL LOAD-DEFORMATION CURVES FOR R/C COLUMNS [98]



(a) Subassemblages

(b) Critical Regions

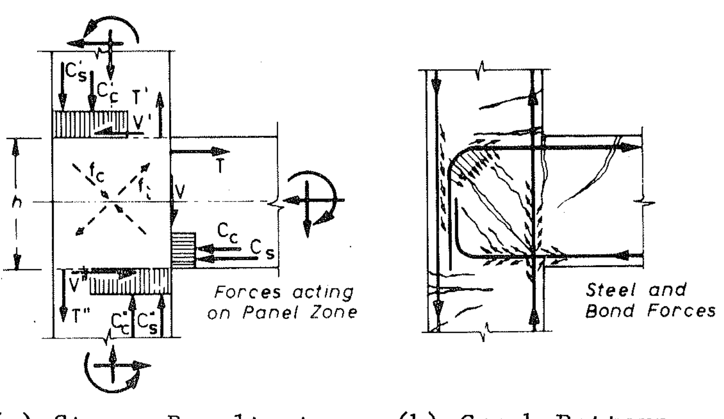
FIG. 19 BASIC SUBASSEMBLAGES OF MOMENT-RESISTING FRAMES AND TYPICAL CRITICAL REGIONS OF THEIR COMPONENTS

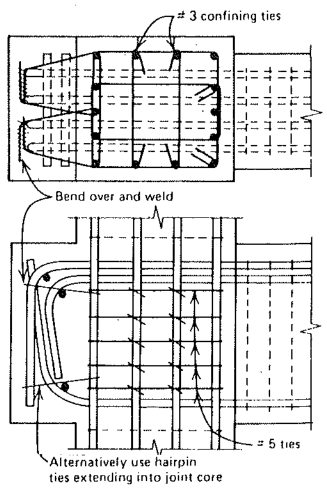


(c) Force Orbit of Square Path Shown in Fig. 18(b)

(d) Envelopes of Shear-Deflection Curves

FIG.18 EXPERIMENTAL RESULTS ON BEHAVIOR OF R/C COLUMNS UNDER 2D LOADING [97]





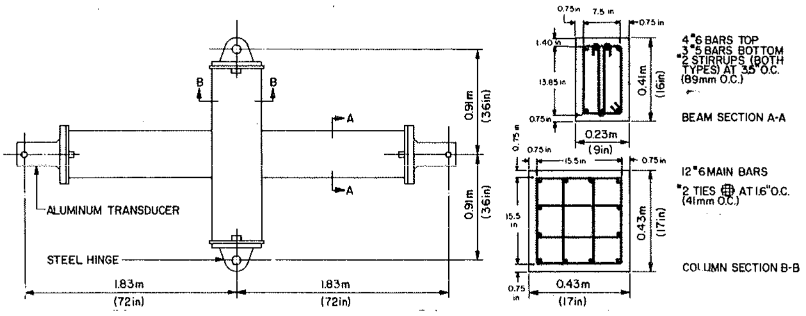
(a) Stress Resultants

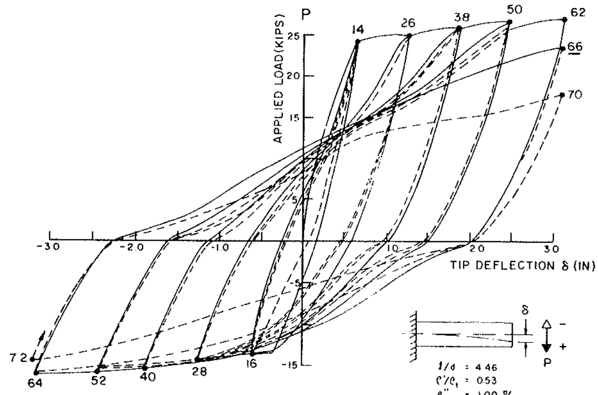
(b) Crack Pattern

(c) Suggested Arrangement of Joint Reinforcement at an Exterior Joint with a Stub

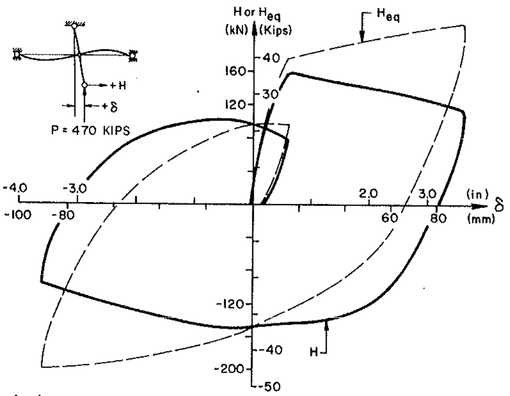
FIG. 20 EXTERIOR BEAM-COLUMN JOINTS [25, 47]

(a) Interior Beam-Column Subassemblage Specimen

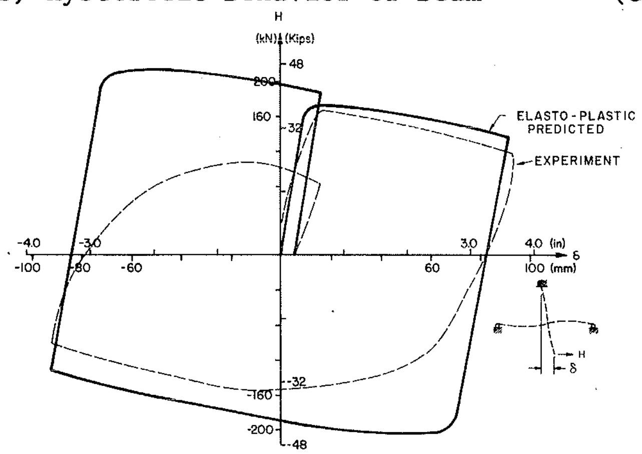




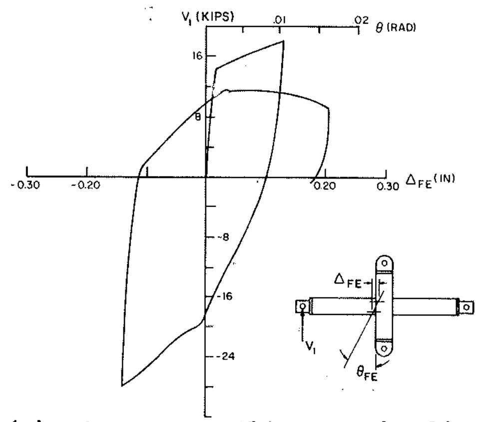
(b) Hysteretic Behavior of Beam



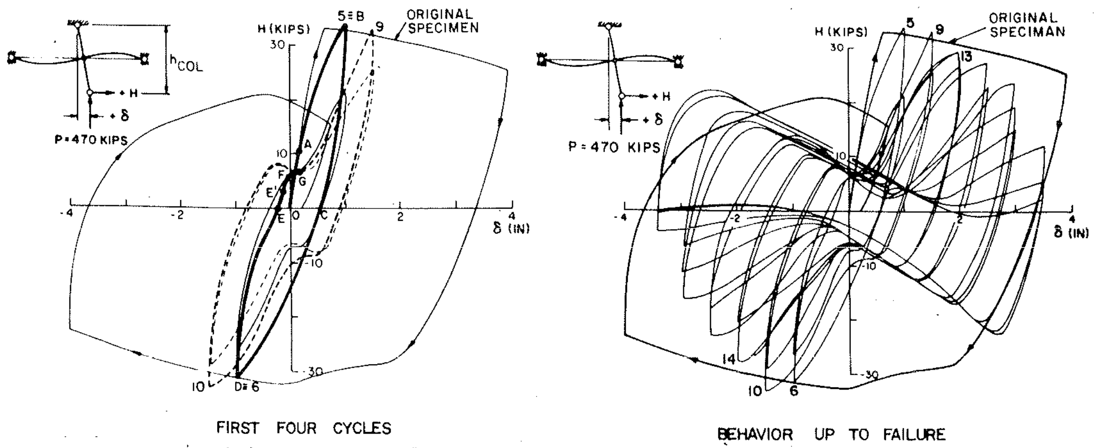
(c) Lateral-Load Deformation of Subassemblage



(d) Subassemblage Experimental and Analytical Curves

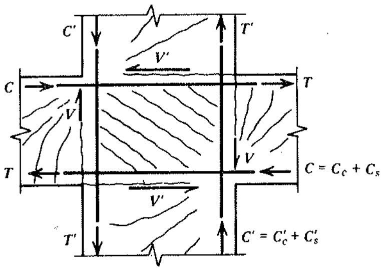


(e) Shear vs. Slippage in Girder's Main Bars of Subassemblage

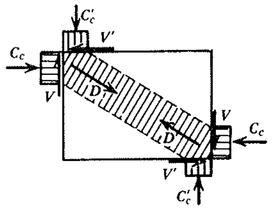


(f) Hysteretic Behavior of Subassemblage

FIG. 21 RESULTS FROM EXPERIMENTS CONDUCTED ON A BEAM AND INTERIOR BEAM-COLUMN SUBASSEMBLAGE [39, 40]



(a) Forces on Beam-Column Joint Core



(b) Shear Transfer by Diagonal Compression Strut

(c) Shear Transfer by Truss Mechanism

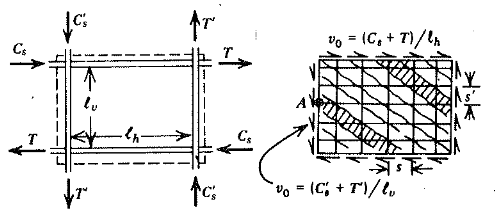
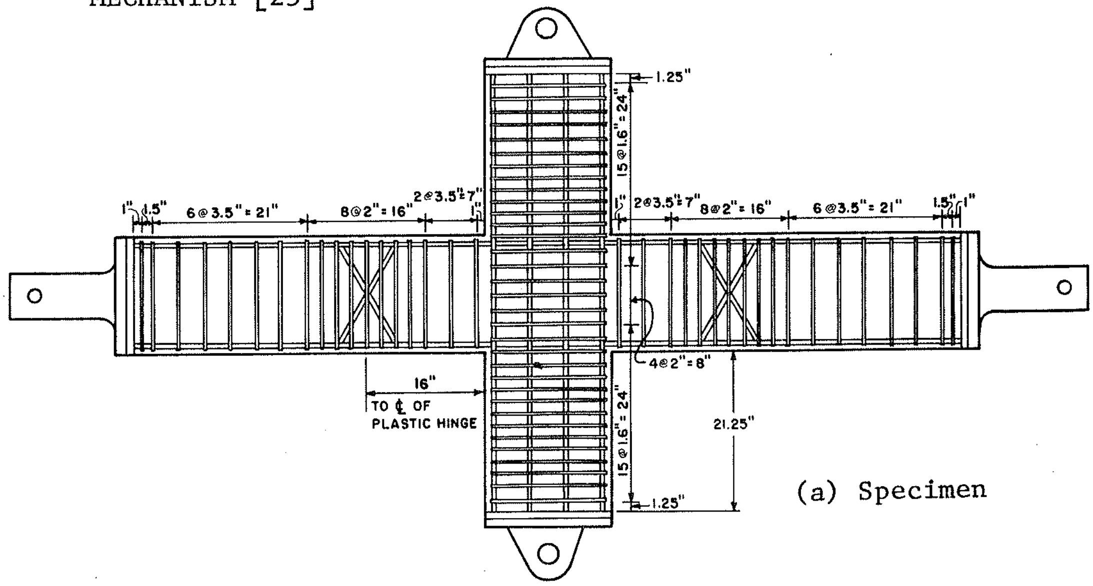
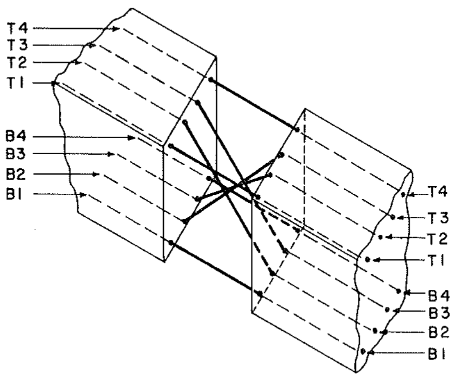


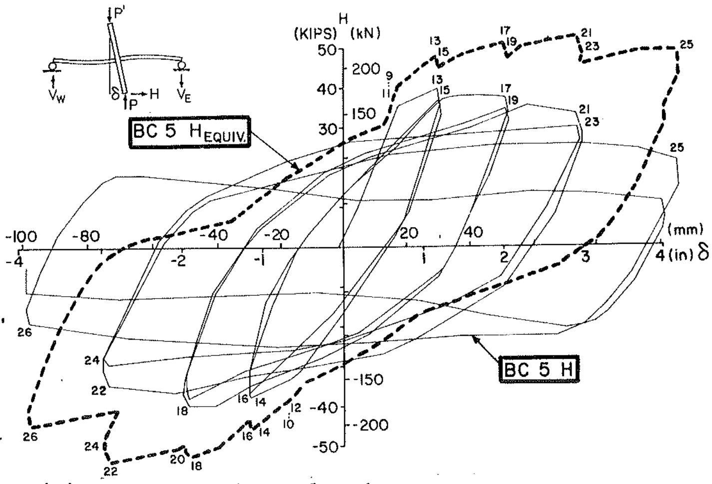
FIG. 22 FORCES ACTING ON INTERIOR BEAM-COLUMN JOINT CORE AND SHEAR TRANSFER MECHANISM [25]



(a) Specimen



(b) Detail of Main Reinforcement at Plastic Hinge Location



(c) Hysteretic Behavior

FIG. 23 BEAM-COLUMN SUBASSEMBLAGE WITH PLASTIC HINGES FORCED AWAY FROM JOINT AND ITS HYSTERETIC BEHAVIOR [119]

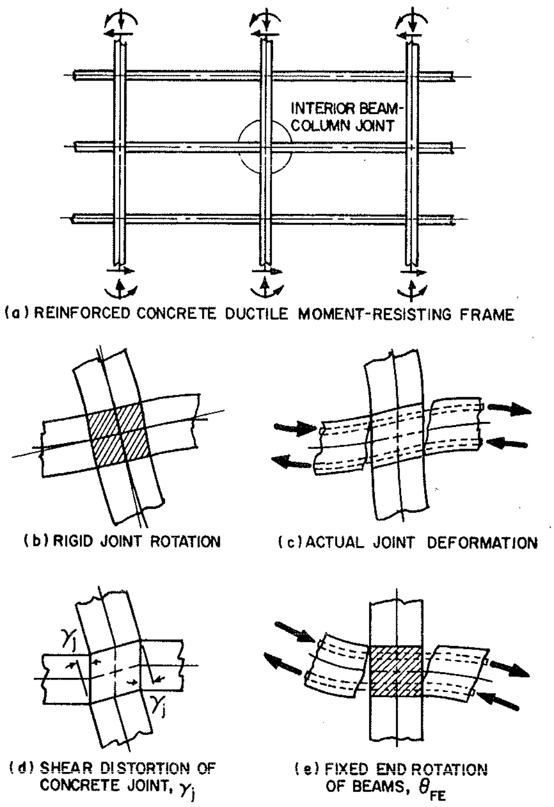


FIG. 24 SOURCES OF DEFORMATION IN AN INTERIOR BEAM-COLUMN JOINT OF A DUCTILE MOMENT-RESISTING FRAME

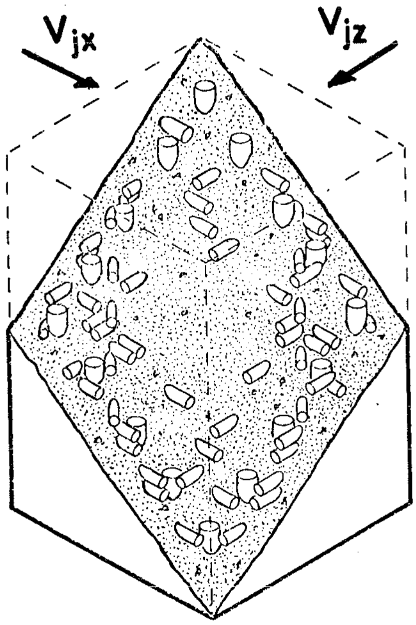


FIG. 25 ISOMETRIC VIEW OF CORNER TO CORNER CRACK ACROSS JOINT CORE IN CASE OF DIAGONAL SHEAR [25]

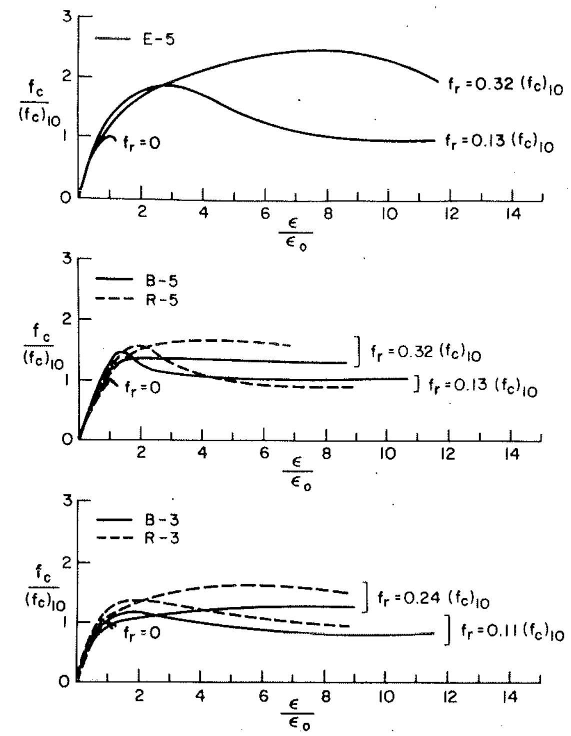


FIG.26 EFFECT OF CONFINEMENT PRESSURE ON COMPRESSIVE STRENGTH AND DUCTILITY OF CONFINED CONCRETE [23]

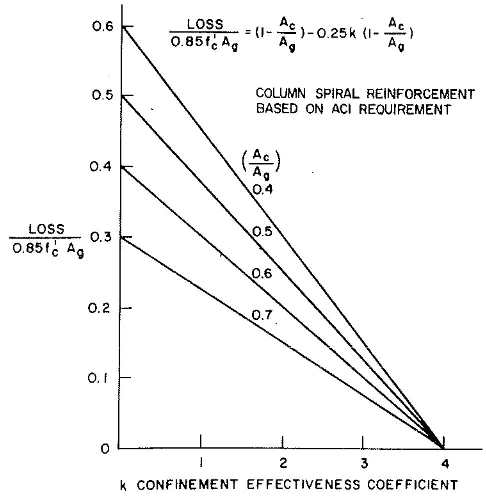


FIG.27 LOSS OF COMPRESSIVE STRENGTH DUE TO SPALLING OF CONCRETE COVER VS. CONFINEMENT EFFECTIVENESS COEFFICIENT [23]

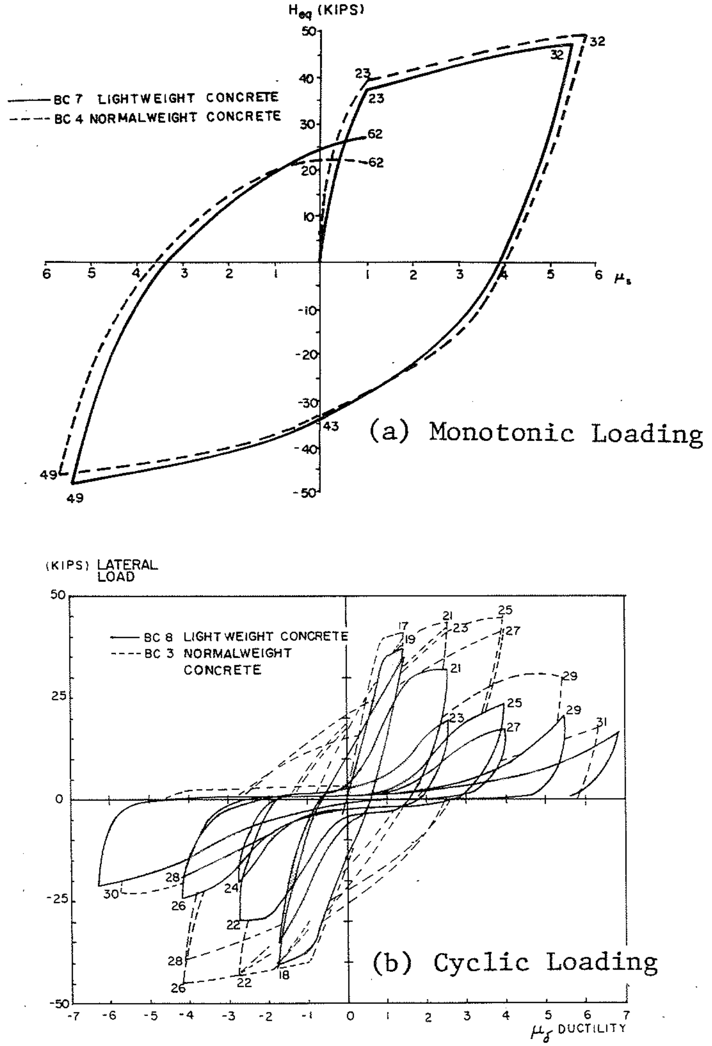


FIG.28 COMPARISON OF BEHAVIOR OF LIGHT AND NORMAL WEIGHT CONCRETE SUBASSEMBLAGES [29]

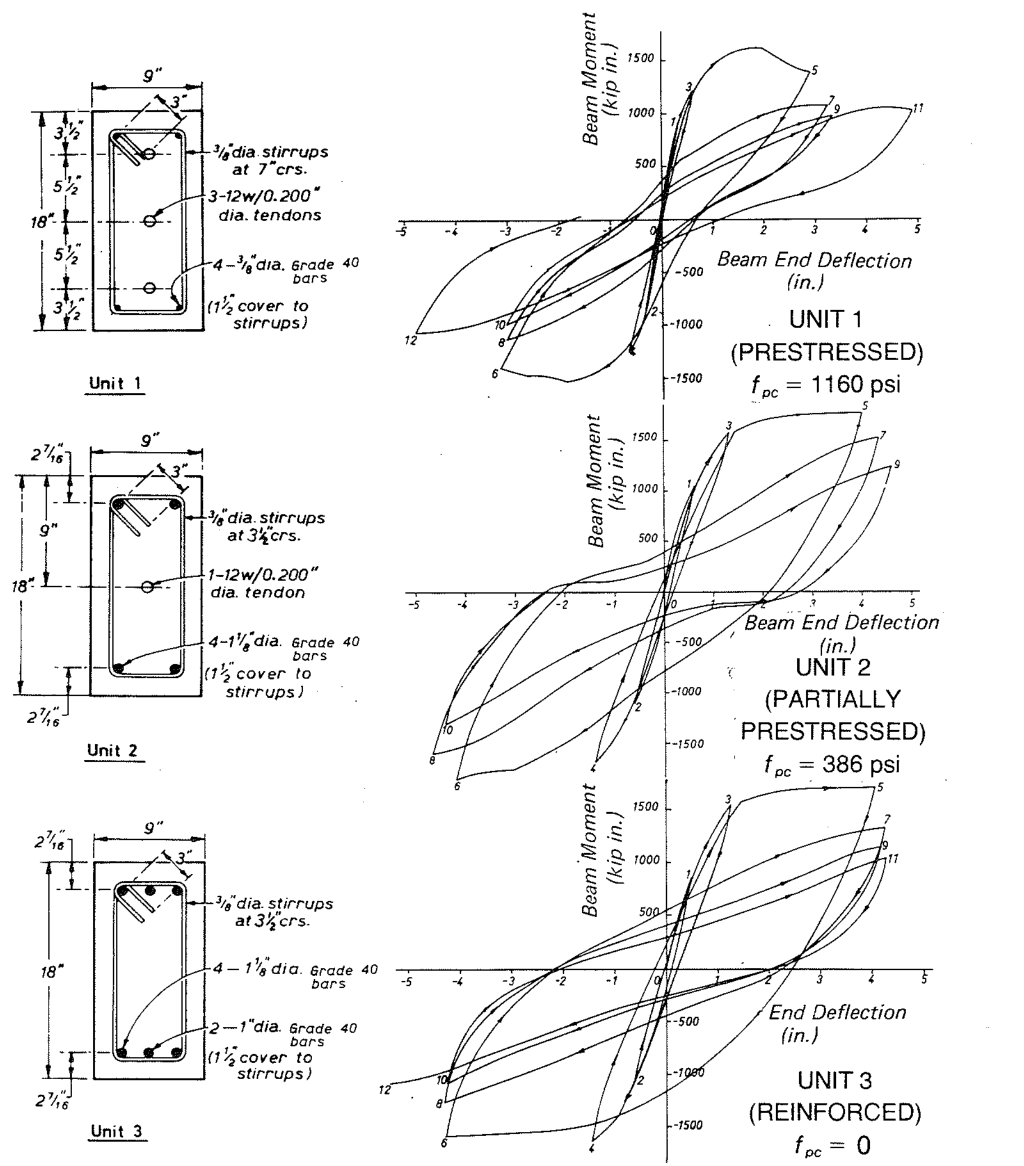


FIG.29 EFFECT OF BEAM PRESTRESSING LEVEL ON HYSTERETIC BEHAVIOR OF BEAM-COLUMN SUBASSEMBLAGE [147, 148]

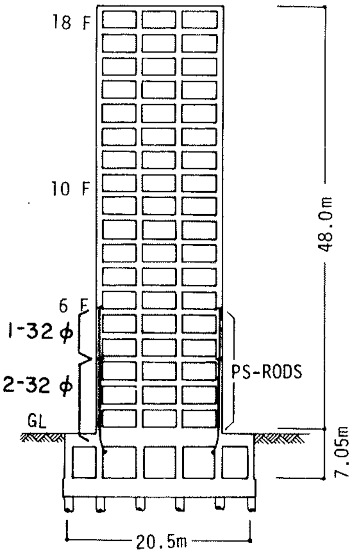


FIG.30 APPLICATION OF COLUMN PRESTRESSING

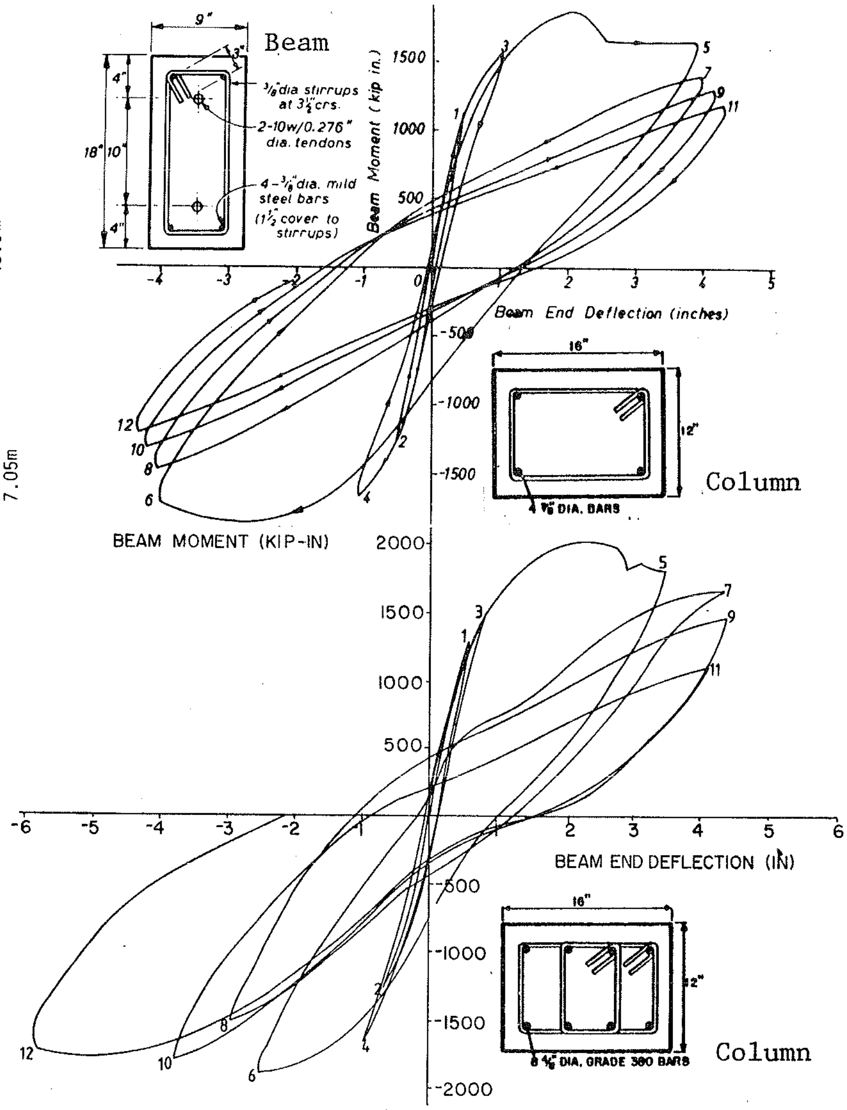


FIG.31 EFFECT OF INTERMEDIATE COLUMN BARS AT THE JOINT CORE ON HYSTERETIC BEHAVIOR OF PRESTRESSED BEAM-COLUMN SUBASSEMBLAGES [148, 152]

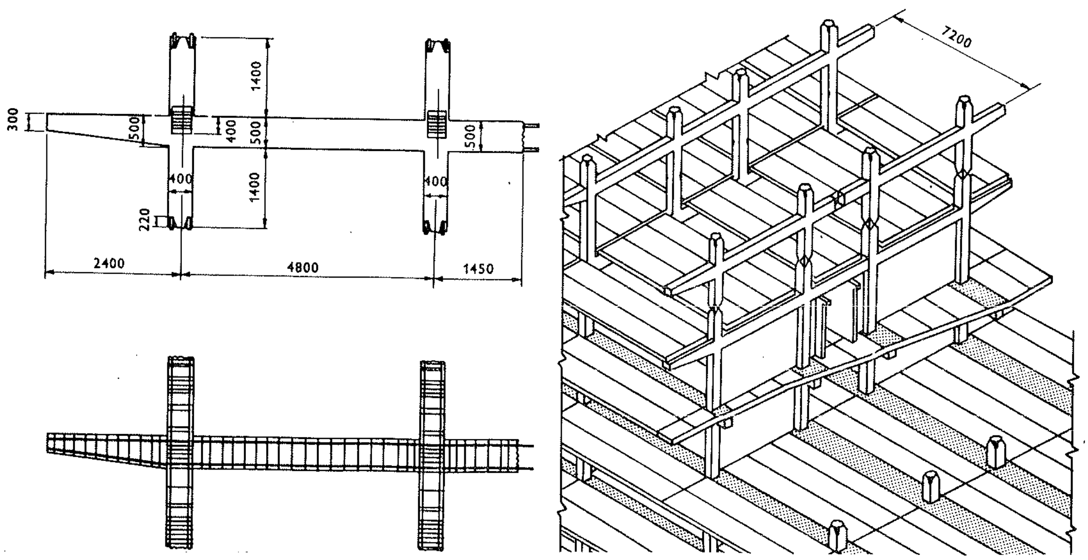


FIG. 32 CONVENIENT LOCATIONS OF JOINTS FOR PRECAST ELEMENTS [150]

THEME III

SEISMIC BEHAVIOUR OF STRUCTURAL CONCRETE TWO–DIMENSIONAL ELEMENTS (Shear walls and others).

COMPORTEMENT SISMIQUE DES ELEMENTS STRUCTURAUX PLANS (murs de contreventement et autres).

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THIS VOLUME HAS BEEN PROVISIONALLY ISSUED WITHOUT THIS S.O.A. REPORT, NOT READY FOR THE MOMENT

THEME IV

SEISMIC BEHAVIOUR OF CONCRETE STRUCTURES: OBSERVATION OF ACTUAL STRUCTURE AND LABORATORY TESTS

COMPORTEMENT SISMIQUE DES STRUCTURES EN BETON: OBSERVATIONS "IN SITU" ET ESSAIS EN LABORATOIRE

Reporter Giuseppe GRANDORI
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INTRODUCTION

The experimental work involved in checking theoretical predictions is highly complex and expensive in earthquake engineering. This is due principally to the following reasons.

1) The purpose behind our work is that buildings, even if severely damaged, should not actually collapse after violent earthquakes. This makes things very complex (e.g. because of the non linearity of the stress-strain curve, or because of the effect of large displacements). Perhaps even more important is the fact that in this field a complete experiment inevitably implies total destruction.

2) The way that a building reacts to an earthquake is very considerably influenced (much more so than in its behaviour under normal working loads) by the collaboration of the non-structural elements, which tend to function upredictably. In fact, for many buildings the design allows for non-structural elements of uncertain durability in time.

3) The violent earthquake that a building is designed to resist also has highly random characteristics. Just one destructive test would not be enough to give complete experimental data on significant variations in these characteristics and their consequent influence on the structure.

4) In current design practice some aspects of the general concept that have great importance for seismic behaviour (the shape of the plan and the vertical cross-sections, besides sometime the position and type of structural elements) are over-influenced by functional and aesthetic requirements. The result is a wide variety of structural types, and unfortunately it often happens that the type chosen is not the most suitable for resisting earthquakes.

5) The non-linearity of the phenomenon means that experiments on small scale models run into almost insurmountable difficulties.

Experimental data is still, despite many years of study, episodic in nature and far from exhaustive. A great effort in international cooperation is called for to sharpen our interpretations of the available data, and, even more, to organise as efficiently as possible the collection of further information.

There are essentially two sources of experimental data: constructions subjected to vibrations produced artificially, in the laboratory or "in situ", and constructions that have actually been submitted to real earthquakes. As to the way that these results are used, here also two fairly distinct categories can be seen. One involves the behaviour of buildings during violent earthquakes when more or less serious damage is provoked and the construction enters the elasto-plastic range. In other words, we here have experiments with a range of final results running from modest but not negligible values right up to total damage leading to collapse.
The second category of results involves the behaviour of structures during moderate (and fairly frequent) earthquakes, which lead to negligible damage or none at all. In this latter case the behaviour of the construction may be considered linearly elastic. For rather obvious reasons, experiments in the elasto-plastic range cannot be carried out by way of artificially induced vibrations on buildings that are actually in service. It can be done in the laboratory, but with considerable limitations that derive essentially from the very high costs. In practice, laboratory experiments in the elasto-plastic range are reserved for studying particular problems or checking the introduction of new design concepts of an explorative nature, such as those carried out by R.W. Clough and A. Huckelbridge (1976) on constructions with foundations allowed to uplift.

A great deal of information can be obtained from observing the damage produced by real earthquakes, even if, for obvious reasons, this cannot be systematically prepared. However, this has always been the most important source of data on behaviour in the elasto-plastic range. C.W. Pinkham and D.F. Moran (1973) observe that: "History has shown that the greatest impetus for improvements in earthquake-resistant design is provided by actual earthquakes. Between earthquake occurrences, progress is dependent on research and studies of past earthquakes. The tendency is for interest to lag and to decrease rapidly following a shock. Regardless of the amount of research and studies, the earthquake provides the ultimate test. Mistakes in judgement, faulty theory, and poor construction practices are located easily by the earthquake".
Experimental work with artificially induced vibrations to study behaviour in the elastic range is relatively easy. Furthermore,

a great deal of information is already available in the literature (practically all of it coming from the S. Fernando quake of February 9, 1971) on buildings that had been equipped with "strong-motion" recorders, and that received shocks of moderate intensity.

OBSERVATION OF THE ACTUAL STRUCTURES

Preamble

For reasons of space, a report of this kind obviously cannot pretend to set out and discuss in detail all the results obtained from the observations of the behaviour of buildings during violent earthquakes. So this chapter will first offer a summary of some methodological questions behind the observations of the effects of earthquakes. Then, as examples, some lessons drawn from certain research groups working on specific cases will be discussed.

Methodology

A preliminary question concerns the choice of buildings to be studied. The general principle is to devote most research work to modern constructions designed according to seismic codes. Obviously these buildings offer the best facilities for checking whether present design criteria are adequate or not. Within this category of earthquake-resistant buildings, priority is naturally given to constructions of a certain importance erected in the most severely affected area, and to buildings equipped with strong-motion recorders. It is obviously important that the original designs should be available and that the proprietor should be prepared to collaborate by giving information on any possible modifications that may have been carried out.

In exceptional cases (buildings with instruments already installed) valuable and complete information can be obtained on the motion of the foundations during the main shock. In all other cases it is essential to gather information on ground motion not just through damage analysis, but also from independent sources. This is the only way to establish a useful experimental correlation between the intensity of the shock and the quantity of damage. For this purpose, data relating to the main shock, obtained from a permanent network of recording instruments, must be integrated with information on the after-shocks obtained from a widespread system of immediate intervention, with instruments (so far as the analysis of the buildings is concerned) located at the foot of the chosen building. If the after-shocks are recorded with instruments that register both acceleration and velocity, then information can be obtained on the laws governing attenuation, on local amplifications and on frequency contents. All of this will make it possible to work back, with considerable reliability, to the characteristics of ground motion during the main shock at the foot of the building involved. It is clearly essential that this emergency service should be prepared and kept efficient by a permanent organisation. Only in this way could the requirements of speed and efficiency be satisfied. International cooperation in this field could lead to organising a service of this kind over fairly vast areas.

The analysis of the behaviour of a construction, and the damage survey, will often depend on the characteristics of the building in question. Anyway, the various inspection teams should always be given common guide-lines, already worked out by experts in this field, partly because any information obtained should be based on homogeneous criteria, but also because the people entrusted with the actual work of inspection may not have specific experience in surveying the effects of violent earthquakes. The most complete contribution for the preparation of guide-lines of this kind was presented by C.M. Duke et al. (1975) to the "Learning from Earthquakes" Panel of the U.S. National Conference on Earthquake Engineering, Ann Arbor. This is an ensemble of Field Guides which, so far as building inspection is concerned, is divided into brief chapters, e.g. Lateral Force Systems, Irregular Systems, Overturning, Moment Resisting Space Frames, etc.. Each chapter contains an introductory commentary which, necessarily very succinct, "is intended to summarize lessons from past earthquakes, current design philosophy, and subjects for which there is an urgent need to gather more performance data". The check lists for some of the chapters are given here as examples.

Lateral force resisting systems:

1. Architectural and structural concepts and their relationship.
2. Redundancy, whether logical system or otherise.
3. Relative behaviour of different systems in the same general area.
4. Relative behaviour of similar systems in different intensity zones.

Irregular systems:

1. Irregular plans and setbacks in elevation.
2. Changes in the lateral load resisting system, in material, masses or stiffnesses.
3. Evidences of torsional response.
4. Relative behaviour of regular and irregular systems in the

same general area.
5. Good and poor design details and constructions procedures.

Overturning:
1. Tension cracks in concrete columns.
2. Damage at splices of steel columns.
3. Are columns offset at splices, indicating possibility of lift-off and not coming in same place.
4. Damage to beams, girders or shear wall elements which indicate up-lift of columns.
5. Evidences of uplift or compression failures between columns and footings or between footings and ground.
6. Tension or compression failures of piles.

Moment resisting space frames, General:
1. Observe behaviour of frame as a whole, with particular attention to failure modes, signs of distress, loading variations, types of connections, and inelastic behaviour.
2. Structural damage caused by deformation affecting adjacent elements.
3. Damage to non structural elements such as infill walls, stairs, and partitions as well as their influence on structural damage.

In the general recommendations at the beginning of the specialised chapters great insistence is laid on the fact that "each destructive earthquake will probably present opportunities to relearn old lessons and, hopefully, to learn some new ones. It is essential to make the most of each opportunity. Investigators must always be on the lookout for new lessons not covered by the Field Guides".
The chapter on "Statistical Data" is particularly important. It discusses the best way to obtain the maximum information from a quantitative point of view on the amount of damage. Layouts are suggested for grouping damage levels, types of buildings, and the shock intensities that presumably affected each building, into categories. For the latter, it is suggested that reference be made to the modified Mercalli scale. These layouts do not substantially differ from the more fully illustrated proposal of R.V. Whitman and others (1975), which introduces the concept of a Damage Probability Matrix.

As to the use of the MM scale, there is really very little choice. This is because historical information, expressed in terms of instrumental data, is almost totally lacking. However, it is generally agreed that something must be done to overcome this limitation for the future. A new formulation has to be prepared to deal with this problem from now on and this should be born in mind when collecting statistical data. The whole subject deserves further attention.

As everyone knows, the MM scale makes use of damage to non-engineered structures, which then become instruments for measuring intensity. Inevitably, these measurements are very widely scattered, which leads to great uncertainty in the application of the MM scale, even for predicting the behaviour of non-engineered structures. This uncertainty becomes even greater when trying to correlate MM intensities with the behaviour of modern engineered structures and their wide range of dynamic characteristics. Furthermore, a Damage Probability Matrix defined in terms of MM Intensities on the basis of damage due to one earthquake is of very little use for predicting damage due to other earthquakes at other sites.
Damage description depending on the maximum ground acceleration, but in all other respects similar to the one just mentioned, was proposed by G. Grandori and D. Benedetti (1973). But it must be pointed out that the maximum acceleration is meaningful only if information is available on the shape of the response spectrum. The authors assumed as an hypothesis that the shape of the response spectrum would be known. This might seem reasonable for a methodological study, but in reality it almost never happens.
A further development of the Damage Probability Matrix that looks promising for the future consists in making the damage depend on two or more characteristic parameters of the earthquake. The parameters that come to mind straight away are the maximum ground acceleration, the maximum velocity, and how long the quake really lasted. This further underlines the need for sufficient instrumental data to work back to the main characteristics of the ground motion at the foot of each building involved.

Typical Lessons From Past Earthquakes

Past earthquakes have taught seismic engineers a number of lessons that can be collectively summarised as follows. Reinforced concrete structures can successfully resist even violent earthquakes provided that certain conditions are satisfied. The design should respect present codes now in force for highly seismic zones, and should be based on an adequate dynamic analysis. The architectural layout should be compatible with a simple and efficacious arrangement of the resisting structures. The details of the metal reinforcing and, above all, of the connections, should be carefully studied. Good engineering judgement should govern the overall design. In other words, seismic codes can lead to first class results, but only if the design itself is of high quality in all other respects.
A general idea on the average usefulness of earthquake codes for a given technological background may be obtained from the

Conclusions of P.C. Jennings and G.W. Housner on the S. Fernando Earthquake of Febr. 9, 1971. "From the experience in this earthquake it appears that ground accelerations with and amplitude of about 15% g mark the threshold of serious damage for most poorer, old pre-1933 buildings, and accelerations of 30% g or greater are associated with very hazardous damage and collapse of most older structures."..."In general, modern structures designed according to the minimum requirements of the codes received, only architectural damage in areas where the accelerations were 20% g or less. There was minor to appreciable structural damage in buildings subjected to shaking in the 20-30% g range, and the damage to buildings of minimum design varied from appreciable to collapse in the area of very strong shaking (30-50% g)"(1).
But the results for individual buildings vary widely from the average behaviour. There were many examples of well-designed concrete construction in the region of strongest ground shaking in S. Fernando that survived the earthquake without significant damage. Jennings and Housner (1971) go to point out the difference in "the performance of the newer (1938, 1949, 1950) buildings at the Veterans Hospital and the main buildings (1965) at Olive View Hospital. Both of these building groups were in the zone of strongest shaking, and were nominally designed to resist essentially the same lateral forces. The Veterans Administration buildings survived the earthquake successfully, those at Olive View did not".
The heart of the matter has been pithily summarised by K. V. Steinbrugge, in the volume edited by R.L. Wiegel (1970): "Too often all efforts are directed toward just meeting the minimum earthquake standards of a building code; just meeting these code provisions is, in reality, placing a building on the verge of being legally unsafe".
On the other hand, it should not be forgotten, as R.V. Whitman and others (1975) stress, that designing for increased seismic forces leads to only modest variations in the intensity of ground motion that would first yield a building and in the damage predicted at various intensities.
An improvement in the situation could probably best be obtained (and at less cost) by raising the overall quality of designing rather than just increasing the lateral force coefficient. It has to be admitted, though, that this general improvement in quality would call for a much longer and more difficult process

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(1) The reference to ground accelerations is of course meaningful only for the specific earthquake in question, with a response spectrum that showed accelerations in a band of maximum amplifications within a range of about T_o=0.1 sec and T_o=0.6 sec. The above conclusions cannot be generalised.

than simply raising a coefficient in the codes.
It is fairly widely accepted by now that something must be done to improve the present state of affairs, whether in terms of an overall raising of design quality or a revision of the lateral forces stipulated in the Codes. More over, to achieve the necessary extra resistance required for special structures such as hospitals, fire stations, etc, Jennings and Housner (1971) recommend "that an importance or occupancy factor be included in the building codes as is the practice in some countries of the world. Such a factor should be applied to the stress or deformation levels at which earthquake motions are to be resisted rather than to the earthquake motions themselves. This approach is suggested because it is thought preferable to first determine the level of earthquake excitation and then to specify which structures should respond with nonhazardous damage and which should be able to withstand the shaking without loss of essential function, i.e., without significant damage.".
Another opinion of Jennings and Housner is worth quoting as a conclusion to these general comments. "The earthquake force previsions of the code have not changed substantially in the last twenty years or more. In this time, however, the knowledge of the resistance of materials and the methods of calculating stresses and structural response have improved steadily. The increased knowledge of material behavior and the refinement of calculation techniques have tended to reduce the conservatism in structural design; e.g., allowable stresses are higher, columns have become smaller and spans have become longer. Also, more daring and innovative structural configurations have been made possible.
However, these advances have not been accompanied by a corresponding refinement in the assessment of the earthquake forces, which are greater than specified by the code, and as a result, the balance that may have existed between these two features of the building code has gradually been tipped in the wrong direction for some applications. (...). In the present state of affairs, as materials and methods of analysis improve, the situation for buildings designed just to code standards deteriorates rather than improves, and the designer who does not understand the true level of earthquake response is given a false sense of security by the increasing refinement of his structural calculations.".
It is much more difficult to summarize the detailed observations that have been made on individual buildings and would require more space than is available. However, some general idea can be obtained indirectly by looking at the recommendations of various authors. Some of the more important are quoted here, but without making any claim to being a complete survey of the field.

- Concrete members should be designed for greater ductility with closer spacing of tie bars, where large bending moments and strains may be incurred.

- Evidence indicated that the effectiveness of tied columns would increase materially if the spacing of the ties was close enough to intersect any possible crack formation and to provide confinement, and if the closure of the ties was anchored sufficiently into the confined concrete core.

- In order to avoid the occurrence of brittle failures, all members should be designed for a shear stress based on the ultimate moment capacity of the member.

- Careful attention should be given to the location of reinforcing bar splices for these can have an influence on damage.

- The design of shear walls should be reviewed and revised to improve the ability to survive strong ground shaking without severe cracking and local failure.

- Provide extra strength and design considerations for shear walls around wall openings.

- Provide either adequate strength or adequate flexibility for linking members between shear walls.

- The deformations resulting from inelastic shear wall behavior also imposes high shears and moments on columns, that should be designed to resist them.

- Codes must require an adequate analysis of the internal elements of shear walls. The "vertical shear" caused by overturning forces through spandrels must be analyzed and provided for.

- The design assumptions must be consistent with the actual conditions and details of construction. Nonparticipating walls should not impinge on the action of the resisting frame element when displaced by the maximum ground motion. Additional stirrups and ties are needed at the ends of columns and girders where infill walls cause high shear stresses.

- Separation joints must be sufficient to accomodate maximum displacements in order to avoid pounding of one building element against another. Prohibit the inclusion of non structural material in the joints without taking into account the resulting reduction of effective joint width.

- Construction joints have been a recurring point of weakness in all earthquakes. Keys (and possibly diagonal reinforcement) to carry the entire shear force must be provided.

- Lightweight concrete, when used for columns and beams, appears to shatter badly when overstrained and, therefore, special reinforcement should be provided. When lightweight concrete is used for floors it should not be run through highly stressed shear walls for this leads to zones of weakness.

- Architects must provide room and facilities for an adequate lateral force-resisting system. Structural engineers, at best, can alleviate only slightly an inappropriate basic layout.

- Code forces should be reviewed and design requirements considering vertical accelerations should be studied.

LABORATORY TESTS

The number of buildings actually subjected to real earthquakes will always be much greater than the number of buildings submitted to full scale laboratory tests. Nevertheless, the laboratory test always gives much more information than the analysis of a single building after a real earthquake. One reason for this is that in the laboratory the input can be programmed to the most suitable level of intensity for the test building, depending on the nature and scope of the research. Also, of course, a much more sophisticated system of instruments can be used.
It is also true, of course, that even buildings "in situ" can be (and have been) equipped with instruments, ready for real earthquakes. But the instrumentation can never be as sophisticated, and is generally less reliable. Furthermore, and this is the main drawback, the cost is in any case very high, while the probability of a suitably equipped building being struck by a violent earthquake within a reasonable period of time, is fairly low. For example, in the Los Angeles area at the time of the S. Fernando quake (1971) there were 66 high-rise buildings equipped with strong motion accelerographs. Not even one of these buildings was appreciably stressed beyond the yield limit. Very useful information was obtained on the "elastic" behaviour of these buildings, but not on the possible damage mechanisms in the non linear range.
So in spite of their limitations, laboratory experiments are still irrepleceable for research work.
Of course, laboratory work with an earthquake simulator able to cause the collapse of a full scale building is the optimum. It is for this reason that for some time now a great deal of effort has been devoted to studying and making experimental equipment of this kind. At the moment there are many laboratories with small and medium sized simulators, while a few institutes are working on the problem of even larger facilities.
These simulators offer the obvious advantage of being able to submit the structure or model to a "real" earthquake, thus

making it possible to measure the effective distribution of the forces and displacements and to observe the damage mechanism. The ultimate limit for possible tests with the small or medium earthquake simulators that are as yet available depends on the fact that the models tested must also be of small or medium size. But unfortunately, as everyone knows, dynamic tests on models of this kind in the non linear range in compliance with the requirements imposed by the laws of dimensional similarity is difficult. Even the simulator installed at Berkeley, for example, has its limitations. It can test building components of up to two floors in height on a full scale. But the velocities and displacements that it can provoke are still insufficient to take the tests up to very severe levels of damage to the structure. Furthermore, as V.V. Bertero and R.W. Clough (1977) have pointed out, "none of the existing medium-scale facilities can be used to carry out studies of the behavior of actual soil-building systems. (...) It would be highly desirable to develop a facility which would permit testing of structures weighing up to 19620 KN and capable of developing velocity in the horizontal direction of up to about 150 cm/sec. A simulator of this type would facilitate investigations on soil-structure interaction since it will permit large numbers of soil layers to be built up on the shaking table. (...) If one or more of these large earthquake simulators becomes available in the near future, its use will still be restricted to proof-type testing and to the study of specific problems such as soil structure interaction under actual seismic excitations for which no other facilities are available. For most structural types, parametric studies of their mechanical behavior may be more efficiently carried out using large-scale loading facilities, since the use of earthquake simulators is not only very expensive for such studies, but, as in the case of dynamic testing, has the basic disadvantage that the input motion and/or the recording instruments have a high probability of malfunctioning due to their complexity. These limitations, coupled with the difficulty of observing the sequence of damage during any test due to its short duration, indicate that it would also be convenient to have other facilities available in which the dynamic excitations are replaced by equivalent pseudo-static excitations."

These pseudo-static facilities offer many advantages for experimental work. The instrumentation is simpler, tests can be suspended to check the instruments and examine the results obtained up to that point, the conditions of the structure can be fully recorded before passing on to the next phase, and a better possibility is offered for understanding the mechanisms leading to deterioration and collapse. The main disadvantage

consists in the fact that, since the load sequence is applied slowly, the effects of the strain rate (essentially the damping and the variations in the yield limit) are shown as negligible. So far as this latter point is concerned, it is worth remembering that a great deal of experimental work has been done on the influence that the strain rate has on the hysteretic behaviour of reinforced concrete structures. V.V. Bertero and R. W. Clough have summed up what is so far known in this field (1977). "From the results obtained to date, it can be concluded that the principal effect of an increase in strain rate on the hysteretic behavior of reinforced concrete flexural regions subjected to different magnitudes of shear and axial forces is to increase the moment capacity at first yielding of the reinforcement. Although studies of the effect of strain rate should be continued to determine the actual increase in strength, to avoid economically undesirable overconservatism in design and construction, according to the results available it is clear that comprehensive experimental studies utilizing pseudo-static testing procedures can be carried out on the behavior mechanisms for stiffness and strength degradation as well as for failure of critical regions of reinforced concrete elements subjected to severe seismic actions. However, some precautions should be noted. In interpreting the results so obtained it is necessary to recognize that although neglecting the observed increase in flexural capacity is conservative from the point of view of the bending capacity design, it is not so for detailing of the reinforcement required to resist the shear or axial forces that can be developed in the same regions or members. Moreover, the effect of strain rate on bond deterioration and on the behavior of the anchorange and splicing of the main reinforcement should be investigated."
Bertero and Clough (1977) also give a detailed review of present trends in laboratory work, as well as a description of the main kinds of equipment in use today. They also give the main results that have so far been obtained, and include an ample bibliography on work in this field.

Some other interesting results have been obtained by a number of different research workers using variable frequency exciters for forced vibration tests. One set of tests has been described by S.A. Freeman, K.K. Honda and J.A. Blume (1977), who say that, "the results of the testing programs have demonstrated the time- and amplitude-dependent nature of the dynamic characteristics of the reinforced concrete structures. Prior to the high-amplitude testing, measured fundamental periods of vibration ranged from 0.37 second to 0.55 second for the bare frame structures. When nonstructural partitions were installed, periods were reduced in some cases to less than 0.30 second. During the high

amplitude testing, where yielding and damage were induced, the period lengthened to 0.9 second. Prior to the high amplitude tests, damping was approximated at values ranging from 1% to 3% of critical for the bare frame and up to 5% with partitions. The high amplitude testing indicated damping up to 4% of critical for the bare frame of structure".
Studies of this kind have also been carried out by others. R. Shepherd and P.C. Jennings (1977) have published a review of such studies. They show that one of the most interesting aspects of resonance testing consists in the fact that for these tests the effects of soil-structure interaction can be measured. Shepherd and Jennings draw the following conclusion. "Reinforced concrete shear-wall buildings, reinforced masonry buildings, and other structures such as in-filled frames which respond to ground motion as shear-wall structures show a much larger degree of soil-structure interaction than has been observed in steel-frame or concrete-frame buildings. The data are too few to draw definite conclusions, but it does appear fairly clear that when such structures are founded on alluvium, the amount of soil-structure interaction can be large enough to influence significantly the earthquake response. Interaction of this amount should be considered in design, as the periods, deflections, and drifts can be affected substancially. The amount of soil-structure interaction and the details of the rotational and translational compliances of the foundation are recommended as central parts of future tests of reinforced concrete buildings".

A new experimental method has recently been developed at the Institute of Industrial Science, University of Tokyo. It is the computer-actuator on-line system, and had been described by T. Okada (1977) as follows, "a principle of the simulation by the computer-actuator on-line system is to solve the non linear differential equation expressing the earthquake response of the structural system to the earthquake ground motion by the computer taking into account the real restoring force characteristics obtained by the pseudo-dynamic loading test performed in parallel with the computer analysis."
This technique is so new that only a limited amount of preliminary experimental work has yet been done.
For an up-to-date summary of research and development needs, see the Final Recommendations of the Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, organised by V.V. Bertero (1977). Some of the more significant recommendations are:

"A careful evaluation of the advantages and limitations of a large-scale earthquake simulator should be performed. This evaluation should consider the possible alternatives to such a large-scale simulator, the costs of such a facility, and the best way for the simulator to be used by all segments of the engineering profession."

"With the increased need for data describing the response of structures under complex loadings, laboratory facilities will need to be improved. Such improvements include the construction of structural floor-wall reaction systems with the capacity to develop multidimensional loads which can be applied to large-scale multistory structures. Laboratories should be encouraged to examine the desirability of computer-actuator on-line system which have the capability of using computers to control the loading applied to the structure as a function of both a specified earthquake ground motion and the structural response. On-line systems will also permit an evaluation of loading histories for less complex testing arrangements. Several facilities should be developed to permit more extensive parametric studies. The different laboratories should correlate and confirm findings and exchange methodologies."

"In order to realize the maximum benefit from research conducted at various institutions, it is desirable that results be presented whenever possible in terms of unambiguous parameters. All too often researchers present results in terms of different parameters without providing sufficient information to allow comparison.

One parameter of particular concern is ductility. While ductility is a useful concept, it has a precise definition and quantitative meaning only for the idealized case of monotonic, linear elasto-perfectly plastic behavior. Its use in real cases where behavior significantly differs from this idealized case leads to much ambiguity and confusion. It is thus difficult to make valid comparisons of "available" ductility values reported by different researchers because they are often based on different response parameters or on yielding values determined using different and/or unexplained definitions.

These experimentally obtained "available" ductility values are also often misused in analytical studies of the "demand" or "required" ductility due to the difficulty of establishing realistic values for the "linear-elastic stiffness and yielding strength." Attempts should be made to integrate the definitions of response parameters that are used in experimental test programs and in analytical investigations.

Furthermore, it is highly questionable whether the performance of different building systems can be properly described and evaluated on the sole basis of elastic stiffness, yielding strength, and ductility. Consequently, there is a need to introduce additional parameters for describing the total hysteretic energy dissipation, number of cycles of reversed deformations, and the degradation in stiffness and strength that has been observed under seismic conditions."

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THEME V

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF CONCRETE STRUCTURES IN SEISMIC REGIONS.

RECOMMANDATIONS POUR UN CODE SISMIQUE DE CONCEPTION CALCUL ET EXECUTION DES OUVRAGES EN BETON

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SUMMARY

The need to implement, in international and national codes, present knowledge on seismic engineering and structural concrete is well recognized.

This implementation calls for the definition of seismic requirements, performance criteria and design procedures and prescriptions.

A brief review of these problems is presented based on documents recently published and having in mind to complement the CEB-FIP Model Code for Concrete Structures with seismic provisions.

SOMMAIRE

Pour le progrès des codes nationaux et internationaux sur le génie séismique et béton structurel, on doit appliquer le résultat des recherches récentes dans ces domaines.

Pour atteindre ce but, il faut définir les exigences fonctionnelles, les critères de performance et les prescriptions de projet.

Prenant pour base des documents récemment publiés, on présente un apperçu des problèmes indiqués en vue d'établir des règles paraséismiques complémentaires du Code Modèle CEB-FIP pour les Structures en Béton.

1 - INTRODUCTION

The need to complement the CEB-FIP Model Code for Concrete Structures (1) with provisions covering earthquake situations is well recognized.

To design structures that meet the requirements derived from earthquake actions three main types of problem have to be dealt with:

i) Definition of design seismic actions;

ii) Procedures for structural seismic analysis;

iii) Provisions for resistant design (structural design including detailing).

This paper mainly covers the last item.

The Economic Commission for Europe (2) has asked the European Association for Earthquake Engineering to prepare, in collaboration with other international associations, a draft of a seismic model code which could be applied to different types of material and construction. This model code should fit the International System of Unified Standard Codes of Practice for Structures (1).

Special provisions for the seismic design of concrete structures exist in several national codes (3).

Tentative provisions for the Development of Seismic Regulations for Buildings have been recently published in the U.S.A. (4). This very comprehensive document complements, in the field of structural concrete, Appendix A of the American Concrete Institute, ACI Standard 318-77 (5).

The Canadian Standard for the Design of Concrete Structures for Buildings published in 1977 (6) includes special provisions for seismic design. Both in Australia (7) and New Zealand (8) codes for the seismic design of concrete structures are under preparation.

The Fédération Internationale de la Précontrainte published in November 1977 Recommendations for the Design of Aseismic Prestressed Concrete Structures (9).

Documents (4) to (9) are used in this report as main reference.

It would be desirable that the seismic provisions which shall complement the CEB-FIP Model Code for Concrete Structures include all the data necessary to structural design in seismic situations. However, this aim looks difficult to reach within a short delay. The work has to be coordinated with the International Association for Earthquake Engineering, which is particularly concerned with the idealization and definition of seismic actions, and with the European Association for

Earthquake Engineering, which should concentrate on the general methods of structural design.

The reliability of structures in earthquake situations depends on a chain of basic assumptions. The concepts included in this paper are established along the following lines:

1.1 - Use of the General Principles on Reliability for Structural Design (10) and of the Common Unified Rules for Different Types of Construction and Material (1).

1.2 - Definition of the seismic actions according to the Basic Note on Actions, A-08, Surface and Bedrock Seismic Vibrations (11).

1.3 - Computation of the seismic effects: action effects and displacements, by the usual methods of structural dynamics.

1.4 - Definition of the strength, stiffness and ductility of elements and structures on the basis of a general theory of structural concrete using idealized stress-strain diagrams for steel, concrete and bond, which take into account the repeated and reversed character of the actions.

1.5 - Detailing of the elements in order to guarantee that their assumed strength, stiffness and ductility can be explored.

The information available on the items mentioned above is not at a uniform stage of development. This information is particularly scarce as regards the ultimate displacements of elements and structures under repeated alternate loading. Consequently the values expressing ductility adopted in the codes are based on qualitative information. This is one of the fields in which further research is urgently needed.

Although this report concentrates on structural concrete earthquake-resistant design, general considerations on seismic requirements and seismic performance are included. These serve as a guide for the specific provisions on computation of resistance and stiffness, qualification of ductility, and detailing.

2 - SEISMIC REQUIREMENTS

The general requirements of structural reliability expressed in (10) are complemented in order to cover the seismic situations. The following requirements are suggested:

2.1 - Structures and structural elements designed to resist earthquakes should sustain with appropriate reliability seismic actions and keep their resisting capacity after these actions cease.

2.2 - Structures and structural elements not designed to resist earthquakes should maintain with appropriate reliability their structural integrity during and after the occurrence of seismic actions.

2.3 - Structures and structural elements should maintain, with adequate reliability, their serviceability during and after earthquakes.

2.4 - Non-structural elements should not collapse with adequate reliability during and after earthquakes.

2.5 - Non-structural elements should maintain with adequate reliability their serviceability during and after earthquakes.

All the requirements indicate that they should be fulfilled with adequate reliability. This is intended to call attention to the use of the probabilistic approach. The reliability to be associated to each requirement should be defined according to the conditions indicated in (10). Consequently the probability of fulfilling the requirement during the reference time should vary according to the "risk to human life or injury, the number of human lives endangered in the case of failure and the degree of social inconvenience resulting from failure. It should also take into account the amount of expense and effort required to reduce the risk of failure".

The probability of occurrence in a reference time of high values of earthquake actions in different regions are very different. For this reason, in some regions, seismic actions are considered variable actions and in other regions exceptional actions. This classification influences the design procedures. Furthermore, the extreme type probability distributions of the maximum intensities in the reference time (expressed for instance in peak accelerations)(11) have coefficients of variation of an order of magnitude of 0.5. It can be theoretically shown that the randomness of the occurrence of earthquakes generally supersedes the other sources of randomness and that in regions of high seismicity it is not economically feasible to reduce the probability of failure to values of an order of magnitude as small as the one adopted for non-seismic situations.

Economic considerations also require the use of degrees of reliability associated with serviceability limit states considerably smaller than those associated with ultimate limit states. However, experience of past earthquakes has shown that the cost of repair is often very high and that it should be reduced by adequate design and construction.

Requirements 2.1 and 2.2 distinguish structures designed to resist earthquakes from those in which the seismic resistance is disregarded. This is intended to give freedom to the designers in their concepts. However, in a construction, the association of both types of structures should be such that the integrity of the whole is kept. In the same way requirements 2.4 and 2.5 refer to non-structural elements associated with structural elements. The interaction between structural and non--structural elements should be duly considered.

3 - PERFORMANCE CRITERIA

Structural concrete seismic codes cover a limited number of types of structures and structural elements. These types of structures should be defined to minimize the ambiguity of the application of the code.

The classification of the types of structural systems presented in ATC--3 is the following:

i) bearing wall system;

ii) building frame system;

iii) moment resisting frame system;

iv) dual system;

v) inverted pendulum system.

This classification is complemented by the indication of different horizontal systems resisting the seismic actions.

The following considerations are limited to three types of vertical resisting structures: frames, shear walls and dual systems.

The application of the general seismic requirements indicated in 2. to these structural systems allows to derive performance criteria. Some of these performance criteria are general, others are specific of the structural type considered.

3.1 - <u>General criteria</u>

The documents (4) to (9) include special chapters where structural design requirements are listed. According to the terminology adopted in this paper such requirements are called performance criteria. The performance criteria are the expression, in terms of performance, of the structural requirements indicated in (2). At the present stage it is difficult to use agreed definitions of the main concepts and to present them in a completely logical way. This difficulty is apparent in all the mentioned documents and also in the present paper.

In chapter 15 - General Seismic Design Requirements of the draft of the New Zealand Code DZ 3101 (5) the following general considerations are presented:

3.1.1 - Strength method of design – The resistance of the members should be determined according to what is called the strength method of design. (This general assertion is adopted in the CEB-FIP Model Code).

3.1.2 - Alternative design methods – The possibility of using alternative methods ensuring adequate reliability is given. (Principle also adopted in CEB-FIP Model Code).

3.1.3 - Ductility — It is indicated that structures should have adequate resistance and ductility. Two complementary clauses are included:

i) structural systems intended to dissipate seismic energy by ductile flexural yielding shall be subject to capacity design procedures;

ii) adequate ductility and dissipation of energy may be considered to have been provided for, if all primary earthquake resisting elements are designed and detailed according to the code.

The need of ductility is expressed in a qualitative way. However, the response modification coefficients indicated in Table 3-B of ATC-3, as well as analogous coefficients included in other codes are an overall measure of ductility.

3.1.4 - Interaction — The interaction of all structural and non-structural elements should be considered.

3.1.5 - Secondary elements — "Consequences of failure of elements that are not a part of the intended primary system for existing seismic forces shall also be considered".

3.1.6 - Diaphragms — "Floor and roof systems in buildings shall be designed to act as horizontal structural elements, where required, to transfer seismic forces to frames or shear walls".

3.1.7 - Non-decreasing envelopes — "In ductile structures the strength shall not appreciably decrease while large earthquake induced cyclic displacements occur".

3.1.8 - Redistribution — Redistribution of critical bending moments obtained by an elastic analysis is allowed within certain limits.

The design basis of ATC-3 (Chapter 3) are more restrict: "the internal forces in the members of the building shall be determined using a linearly elastic model". No redistribution is mentioned.

Further to the general considerations on design requirements, DZ 3101 includes design assumptions which cover: the creation of plastic hinges in frames, shear walls and bridge piers; the computation method of the flexural strength of the plastic hinges; overstrength capacity of the potential plastic hinges; and the influence of cracking in the deformability of the concrete members.

The serviceability limit state requirement included in the FIP Recommendations for Prestressed Structures (9) reads: "A large increase in the strain of the tendons after cracking of the concrete is possible during an earthquake, and any resulting residual strains will cause a loss of prestress. In order to avoid such prestress losses, the strain in the tendons located in the tensile zone should not exceed the initial tensile strain at that section at the time of prestressing, or the limit of proportionality of the steel employed, whichever is greater". The method for carrying out this checking is indicated. Due to the high variability of seismic actions, this performance criterion, which considerably complicates design, is considered of secondary importance.

The ultimate limit state requirements of FIP Recommendations (9) read: "Methods of structural analysis taking into account the elastic plastic deformation and the ultimate strength of the structure could be used. If post-elastic deformation is permitted, it should be verified that:

a) premature failure of concrete or tendons will be avoided;

b) brittle fracture or failures will be avoided;

c) full integrity of all structural elements and connections and the structure as a whole is assured up to the limit state of the structure.

Furthermore, FIP Recommendations include an accidental ultimate limit state relating to maximum credible earthquake. The advantage of including this third limit state is controversial and out of the general reliability concepts introduced in (10).

3.2 - Materials

3.2.1 - Concrete

Seismic codes reduce the range of the allowable design strength of concrete as compared to usual situations. For instance DZ 3101 indicates that the compressive strength of concrete f'_c shall be 20 MPa $\leq f_c \leq$ 55 MPa . These limitations are not severe. A general limitation based on seismic arguments is difficult to justify. If some limitations are to be introduced they should be related to specific types of behaviour. This is the case of light-weight concrete whose strength is limited by ATC-3 in specific conditions (see (4) chapter 11.5.1).

3.2.2 - Steel

DZ 3101 limits the yield strength of reinforcement in potential plastic hinge regions to 415 MPa. ATC-3 limits the steel qualities of buildings assigned to categories C and D in the following way: "longitudinal reinforcement in special moment frames and in wall boundaries shall comply with ASTM A - 706 ($f'_y \geq$ 415 MPa). ASTM A-615 grade 40 reinforcement ($f'_y \geq$ 276 MPa) may be used in these elements if (1) the actual yield stress based on mill tests does not exceed the specified yield strength by more than 18 000 psi (124 MPa) (retests shall not exceed this value by more than an additional 3 000 psi) and (2) the ratio of the actual ultimate tensile stress to the actual yield stress is not less than 1.25.

The need to introduce these severe conditions is justified in the ATC commentaries. However, in the opinion of the Author, it would have been preferable to present the performance criteria which are in the background of these prescriptions instead of imposing such limitative conditions.

The performance criteria should be:

i) Members should be designed to fail by ductile flexure and not by shear or bond. Consequently, overstrength of flexural member should be limited to avoid the types of failure indicated;

ii) Sufficiently large inelastic rotations shall develop in the plastic hinges. This requires the use of steel with a sufficiently large uniform elongation (not a high value of the ratio of the actual tensile strength to the actual yield stress).

The Author's point of view is that most types of steel used in non--seismic regions can also be used in seismic regions. The designing and detailing should adequately take into account their mechanical properties.

DZ 3101 only allows deformed bars to be used for longitudinal non--prestressed reinforcement. The superior behaviour of deformed bars is recognized. However, to forbid the use of longitudinal plain bars in seismic regions looks too severe. DZ 3101 includes further conditions imposing "the use of grade 275 plain round bars for transverse reinforcement, except that grade 380 plain bars of up to one half the diameter of the longitudinal bars may be used as transverse reinforcement, provided that such plain bars are permanently identified". The reasons for these prescriptions are justified in a commentary. However, they are not completely convincing. The implicit performance criteria are sound; the adopted prescriptions too strict.

3.2.3 - Tendons and ducts

FIP Recommendations (9) include no limitation on the use of prestressing steel based on its quality.

The use of unbonded tendons in aseismic prestressed structures is limited by these Recommendations according to specific prescriptions.

The Australian draft (7) requires the structures in zones of high seismicity to have all tendons fully bonded. Also the ducts carrying post-tensioned tendons through beam-column joints shall be corrugated.

3.3 - Frames

Structural concrete moment resisting frames under intense seismic actions develop plastic hinges. The indicated seismic codes impose conditions on the location of these hinges and on their ductile character.

NZS 4203:1976, (12) under given conditions (entire shear resisted by reinforcement and limitation of axial force) permits the formation of hinges in columns. The redaction of DZ 3101 is more severe: "The flexural reserve strength of columns in frames with more than two storeys shall be sufficient to preclude the possibility of simultaneous plastic hinge formation in the top and bottom of all columns in any but a top storey of a bent". "With the exception of the top storey the likelihood of yielding in columns before the yielding of beams shall be minimized".

Furthermore, it is indicated that "to protect columns against brittle failure, reserve shear capacity shall be provided". Specific rules are given to satisfy this condition.

ATC-3 distinguishes in buildings four performance categories: A, B, C and D. Special moment frames are required for buildings assigned to categories C and D. The provisions for designing and detailing of special moment frames include conditions on the relative flexural strength of columns and beams. The general condition is: "At any joint and in the plane of the frame considered, the moment about the center of the joint corresponding to the flexural strength of columns or column shall exceed that corresponding to the flexural strength of the beams framing in the joint". Complementary provisions cover the cases in which this condition is not satisfied.

The correspondent conditions of the Canadian Code are less severe. The condition – sum of the moment strength of the columns greater than the moment strength of the beams – can be disregarded if "the sum of the moment strengths of the confined cores of the columns is sufficient to resist the applied design loads". Furthermore, "particular beam-column connections at any level may be exempt of the above condition provided the remaining columns and connected flexural members comply and are capable of resisting the entire shear at the level accounting for changes in forces and torsion resulting from the action of the nonconforming connections."

According to the FIP Recommendations (9) in flexural members (beams) "suitable precautions should be taken to ensure appropriate plastic hinge positions and adequate plastic hinge rotation capacities under severe or excessive earthquake loading". The following factors affecting ductility should be taken into account:

a) "The ductility decreases with increasing tension steel content, which should preferably be such that

$$\frac{A_p f_{pd} + A_s f_{yd}}{b\, d\, f'_{cd}} < 0.2$$

where: A_p – area of prestressing steel in tension zone;

A_s – area of non-prestressed steel in tension zone;

b – width of the section;

d – depth from extreme compression fibre to centroid of tension reinforcement;

f'_{cd} – concrete design compressive strength;

f_{pd} – design tensile strength of prestressing steel;

f_{yd} – design yield strength of non-prestressed steel.

"For members with steel placed at various positions in the section, this requirement should be interpreted that at the design moment the neutral axis depth should not exceed 0.25 of the overall depth of the section".

The condition of limiting the steel index to 0.2 is also included in the Australian Draft (7).

b) "At positions of moment reversals, where the greatest ductility requirements exist, the ductility is enhanced if tendons are placed near both extreme fibers rather than axially only".

c) "Confinement reinforcement should be provided at critical sections, especially where there are high moments combined with high shears".

d) "Axial compressive forces greatly decrease the ductility of prestressed concrete members".

e) "The design moment of the section should be at least equal to 1.3 times the cracking moment".

In determining the shear capacity the FIP Recommendations indicate that "the plastic hinge moments should be determined considering the possible overstrengths of the materials and these enhanced plastic hinge moments may be taken as 1.15 times the flexural capacities calculated on the basis of the characteristic strengths of the materials".

To minimize the risk of failure of columns it is indicated that they should be designed on the following basis:

a) "they should have a larger margin of safety than other structural members".

b) "the ultimate shear capacity should be provided as stated above".

c) "adequate flexural ductility should be provided as stated for beams".

All the codes mentioned require ductility as a performance criterion of frames. Some of the codes, as e.g. ATC-3, distinguish categories which correspond to different ductility qualifications. However, further research is needed to an accurate definition of ductility. For the present the existing knowledge expressing the seismic requirements should be implemented in well defined performance criteria. In the near future it should be possible to compute for each design not only resistance and stiffness but also ductility. At each point and in each direction, accurate diagrams relating alternate forces and horizontal displacements, and including the ultimate values of displacements, should be defined.

3.4 - Walls and hybrid structures

DZ 3101 distinguishes two types of shear wall structures: non coupled and coupled by ductile beams.

For the first ones it is indicated the "they shall be designed to be capable of dissipating significant amount of energy, preferably by flexural yielding".

For the second ones "a significant part of the seismic energy to be dissipated shall be assigned to the coupling system".

As for beams, it is recommended that the "dependable shear strength of shear walls is in excess of their flexural overcapacities".

The Canadian Code (6) substitutes the term "shear wall" by "ductile flexural wall" and presents very general performance criteria: "Ductile flexural wall shall be designed to have adequate ductility and energy absorption capacity in accordance with generally accepted principles".

ATC-3 indicates no specific performance criteria to shear walls but a set of reinforcement details and limitations.

The very important problem of openings and coupling of shear walls is only very briefly covered in all the mentioned codes.

The difficulties inherent in the definition of ductility of wall and hybrid structures is even greater than for frames. The ductility factors adopted for hybrid structure are intermediate between those of frames and walls.

To adopt these intermediate factors it is often required that the overall capacity of resistance to horizontal forces be larger than a given fraction of the total force on the hybrid structure. The book by Park and Paulay (13) includes very useful information on the behaviour of wall and hybrid structures.

3.5 - Diaphragms and other structures

The New Zealander draft DZ 3101 (8) includes in the same chapter the seismic requirements for walls and diaphragms. This can be explained by the similarity of their structural shape. However, the special character of the diaphragms derives from their usual function as slab.

The ATC-3 (4) further to shear walls and diaphragms also includes in the same chapter braced frames. The conditions refer to minimum reinforcement and limiting shear stresses.

The specific condition to be applied to diaphragms requires the existence of boundary members where the compressive stress exceeds given limits. The ATC-3 allows the topping on a precast system floor to serve as a diaphragm provided the cast-in-place topping is proportioned and detailed to resist the design shear forces. The ATC-3 includes special conditions to be applied to boundary elements (e.g. for boundary elements around openings).

The FIP Recommendations (9) include considerations refering to pre-stressed concrete piles. Special structures such as shells are not covered by the mentioned seismic codes.

4 - DESIGN PROCEDURES

4.1 - Introduction

The performance criteria presented above should be transformed into design procedures for the checking of serviceability and safety in seismic situations.

According to Level 1 method, for each limit state, the design procedure includes:

a) The idealization of the structural behaviour;

b) The definition of design equations and of the basic variables included in them;

c) The definition of the partial factors of safety;

d) The reliability checking.

The serviceability limit states correspond to the limitation of crack widths and relative displacements. The ultimate limit states correspond to failure. The design procedures for ultimate limit states are discussed first.

4.2 - Checking of ultimate limit states

In reinforced and prestressed concrete structures the ultimate limit states are reached well within the non-linear domain. In usual conditions, failure occurs after the structure has suffered several deformation cycles with large excursions in the plastic zone.

The checking of ultimate limit states should be carried out by verifying that the resistance and ultimate deformability of all regions is not exceeded due to the seismic actions. On the other hand the structure should keep its integrity at the ultimate deformed stage. Consequently the safety checking involves two types of problems:

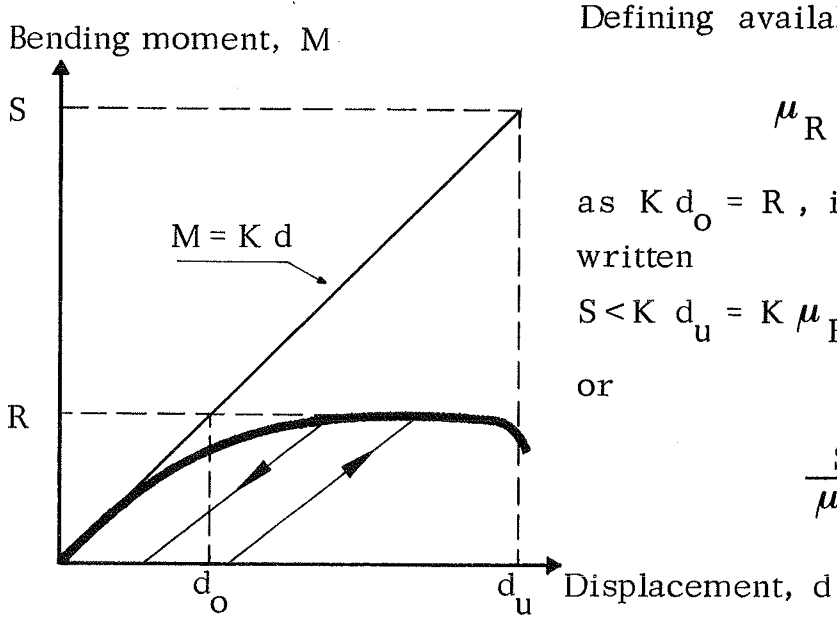
a) Verifying that the deformability or ductility of all sections expected to yield is sufficient;

b) Verifying that the resistance of the sections which should keep structural integrity is compatible with the resistance of the elements allowed to yield.

The design equations which correspond to these two types of problems are different. Furthermore the design equations which correspond to the first verification may be expressed in different ways.

Assume the safety verification of a flexural member. The linear analysis of the structure gives a bending moment S due to the combined action of permanent and superimposed loads and seismic forces. The behaviour of the member is defined by a stiffness K, an ultimate bending moment R and an ultimate displacement, d_u. The elastic displacement which would correspond to the bending moment S is $d_S = \frac{S}{K}$. Under the assumption that linear and non-linear displacements due to the seismic actions are equal, the design inequation becomes:

$$d_S = \frac{S}{K} < d_u \quad \ldots\ldots\ldots\ldots\ldots \quad 1)$$



Defining available ductility by

$$\mu_R = \frac{d_u}{d_o} \quad \ldots . \quad 2)$$

as $K d_o = R$, inequation 1) can be written

$$S < K d_u = K \mu_R d_o = \mu_R R \quad \ldots 3)$$

or

$$\frac{S}{\mu_R} < R \quad \ldots . \quad 4)$$

Finally defining required ductility by $\mu_S = \frac{S}{R}$ and substituting in 4) it comes

$$\mu_S < \mu_R \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 5)$$

Expressions 1), 4) and 5) are equivalent design inequations.
In the first case the design inequation is expressed in displacements and corresponds to the condition: acting displacements smaller than ultimate displacements.

In the last case the design inequation is expressed in ductilities and corresponds to the condition: required ductility smaller than available ductility.

In the intermediate case the design inequation includes bending moments and available ductility and corresponds to the condition: acting elastic bending moment divided by available ductility smaller than resisting bending moment. In this case the reduced bending moment $\frac{S}{\mu_R}$ should not be interpreted as an action alone. The reduction coefficient μ_R depends on the non-linear behaviour of the member and consequently on its resistance.

As indicated, the checking of ultimate limit states involves two types of verifications: a) ductility and b) structural integrity.

The checking of the second type conceptually differs from the first one. The structural integrity is verified by conditions of internal equilibrium.

For instance, after carrying out the verification of the deformability of the flexural members it is necessary to check their resistance to the shear forces which equilibrate the yielding bending moments. In this checking the bending resistance plays the rôle of actions and the shear forces of resistances. The design equation can be written

$$V_{act}\,(R) < V_u \quad \ldots\ldots\ldots\ldots\ldots \quad 6)$$

where $V_{act}\,(R)$ are the shear forces due to the available bending resistance and V_u the resisting shear forces.

The probabilities of failure to be associated with the verifications of ductility and structural integrity should be different. Further studies on systems reliability are needed to allow an improved quantification of these probabilities.

4.3 - Checking of serviceability limit states

In seismic situations the problem of structural cracking is in general disregarded. The serviceability limit states are defined by limitation of relative displacements. The allowable values of these relative displacements are established in order to reduce the damage in non-structural elements and to limit second order effects.

The design inequation takes the form

$$d_{act} < d_{lim} \quad \ldots\ldots\ldots\ldots\ldots \quad 7)$$

where d_{act} are the displacements due to the seismic actions and d_{lim} the allowable ones.

An important problem consists in defining the values of the seismic actions to be adopted in the checking of serviceability limit states. It is often mentioned (9) that this checking should be carried out for moderate earthquakes of an intensity considerably smaller than the characteristic one adopted for checking the safety. This leads to the assumption of linear structural behaviour. Under this assumption there is the freedom of choice of the value d_{lim} taking into account the return period of the characteristic seismic actions. According to the basic note on seismic actions, A-08 (11), the characteristic value (0.95 fractile) of the maximum peak acceleration in 50 years is approximately 4 times the characteristic value of the maximum peak acceleration in 1 year. Consequently the characteristic maximum elastic displacement allowable in 50 years is 4 times the characteristic maximum value allowable in 1 year reference time.

5 - DESIGN PRESCRIPTIONS

5.1 - Introduction

Seismic provisions to be included in the draft of the Portuguese Code on Concrete Structures (14) have been recently prepared (15)*. As the Portuguese Code follows closely the CEB-FIP Model Code, the draft of some of these provisions is presented.

The seismic provisions are of two types. Those of the first type allow to quantify the ductility factor defined in 4.2 and indicate how to carry out the safety checkings. Those of the second type give design procedures and prescriptions on detailing for the different types of structural systems.

The Code on Safety and Actions on Structures (16) introduces the concept of behaviour factor K to be used in the equivalent static method to reduce the seismic forces obtained from the elastic response spectra.

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* The drafting commission of the Portuguese Code on Structural Concrete includes J. Arga e Lima, A. Teixeira Coelho, V. Monteiro and M. Castanheta, researchers of the staff of LNEC.

This behaviour factor K depends on the available ductility of the structural system and on the type of use of the structure. The available ductility depends on the mechanical properties of the materials and on the detailing of the reinforcement. The use of the structure is accounted for having in mind to limit the use of the available ductility in structures for which the seismic damage should be reduced as compared to normal structures.

When the available ductility is fully used the behaviour factor K should be identical to the ductility factor.

5.2 - Behaviour factor, K

5.2.1 - The values of the behaviour factor K are defined in Table 1.

Table 1

Structural system	Normal ductility	Improved ductility
Framed reinforced or prestressed concrete structures.................	2.0	3.0
Walls and diaphragms.............	1.2	1.5
Hybrid structures including walls and frames	1.5*	2.0*

* The values indicated can only be used if the frames are able to resist more than 25% of the total seismic forces. Otherwise the values for walls should be adopted.

The structures which follow the design and detailing prescriptions of the CEB-FIP Model Code are considered of normal ductility. Structures which follow the prescriptions indicated below are considered of improved ductility.

5.2.2 - In buildings and other structures which should suffer reduced damage due to strong earthquakes (hospitals, communication centers, fire fighting facilities, power stations, etc.) the values of K in Table 1 should be multiplied by 0.8.

5.2.3 - In the zone of highest seismicity (zone A of the Portuguese Code) all structures of the types indicated in 5.2.2, should be of improved ductility.

5.3 - Materials

5.3.1 - Concrete

In structures of improved ductility the use of concrete of the quality lower than C20 is not allowed unless there is a special justification.

5.3.2 - Steel

In structures of improved ductility the use of non-prestressed steel of a quality higher than S500 is not allowed, unless there is a special justification.

5.3.3 - Prestressed tendons

In structures of improved ductility all tendons shall be fully bonded.

5.3.4 - Ducts

In structures of improved ductility the ducts carrying post-tensioned tendons through beam-column joints shall be corrugated.

5.4 - Beams in structural systems of improved ductility

5.4.1 - Percentage of the tensile longitudinal reinforcement of beams, with reference to the total area of section, shall not exceed values indicated in Table 2.

Table 2

Maximum percentage values of tensile longitudinal reinforcement, at bottom or at top

Type of steel	S220	S400	S500
ρ_{max} (%)	2.5	1.5	1.0

5.4.2 - Beams shall be provided with longitudinal reinforcements throughout their length both at top and bottom. Each of them shall conform to minimum percentages indicated in CEB-FIP Model Code (1), and consist of at least two bars 12mm in diameter.

5.4.3 - Positive moment strength of beams at the frame joints shall not be less than 50% of the negative moment strength.

5.4.4 - Throughout the length of beam both at top and at bottom, reinforcement shall be provided that is at least one-fourth the largest amount required at joints (in the corresponding face) to resist moments resulting from the combination of actions in which seismic loading prevails.

5.4.5 - Web reinforcement shall be designed to resist a force corresponding to the sum of the shear force resulting from the actuation, at the end sections of the beam, of moments equal to 1.25 times the characteristic moment strengths of such sections, which may be induced by horizontal displacement of the frame, and of the isostatic shear force due to gravitic actions included in the combination of actions in which the seismic action prevails.

5.4.6 - Throughout a length of beam not less than 2d from column face, web reinforcement shall consist of stirrup-ties at maximum spacing of 0.3d or 20cm. First stirrup-tie shall be located at a distance not greater than 8cm from column face.

5.4.7 - For the purpose of designing anchorages and lap splices in bars, these shall always be considered in bad bond conditions. No welded splices at beam ends shall be allowed.

5.5 - <u>Columns in structural systems of improved ductility</u>

5.5.1 - Cross-section of columns shall conform to condition

$$N_{Sd} < 0.6 \; f_{cd} \; A_c$$

where:

N_{Sd} – design axial load corresponding to the combination of actions in which seismic action prevails.

f_{cd} – design value of compressive ultimate strength of concrete.

A_c – area of column cross-section.

5.5.2 - Minimum area of longitudinal reinforcement shall be 0.8 or 0.6% of column cross-section, depending on steel used being S220 or S400 and S500, respectively.

5.5.3 - Maximum area of longitudinal reinforcement shall not exceed 6% of column cross-section even in zones of splicing of bars.

5.5.4 - At each joint the sum of moment strengths of columns under the axial force corresponding to combination of actions in which seismic action prevails shall be greater than the sum of moment strengths of beams induced by horizontal displacement of the structure. For this condition only the confined core shall be taken as the column cross-section.

5.5.5 - Web reinforcement shall be designed to resist a shear force corresponding to actuaction, at the end sections of the column, of moments equal to the moment strengths in these sections that may be induced by horizontal displacement of the structure; and besides by considering an axial force corresponding to the combination of actions in which the seismic action prevails.

5.5.6 - At the end zones of columns, throughout a length from joint faces or foundation elements greater than the largest dimension of the cross-section and one-sixth the clear height of column, spacing between hoops shall not exceed 10cm.

5.6 - Beam-column joints in systems of improved ductility

5.6.1 - In the beam-column joints hoops shall be provided transversely to column axis which at least conform to criteria in 5.5.6. Hoops shall be designed on basis of horizontal shear forces resulting from compression and tensile forces transferred to these zones by beams framing into there, and taking into account shear forces transferred by columns.

The parts of shear force corresponding to forces transferred by beams shall correspond to 1.25 times the moment strengths of such beams that may be induced by horizontal displacement of the structure.

5.6.2 - In joints with beams framing into the four sides of the column, the area of transverse reinforcement in the joint may be one-half that required by 5.6.1 if every beam has a width not less than one-half the column width and a depth not less than three-fourths that of the deepest beam framing into the joint.

5.7 - Shear walls of improved ductility

5.7.1 - Shear walls shall be provided with distributed horizontal and vertical reinforcement in both faces, consisting of bars spaced less than 30cm and in a percentage of more than 0.12% in each direction and at each face.

5.7.2 - Vertical reinforcement designed to resist bending on the plane of wall shall concentrate near edges of shear walls distributed in a zone not smaller than wall thickness. This reinforcement shall be made up of at least four bars 12mm in diameter. At each edge reinforcement section shall not be less than 0.25 percent or 0.15 percent of the total section of shear wall depending on whether steel used is S220 or S400 and S500, respectively.

Concentrated vertical reinforcement thus formed shall be confined by hoops in accordance with provisions for columns.

6 - CONCLUSIONS

6.1 - The design of structures on seismic situations involves chains of decisions intimately related. The safety and serviceability of the structure depends on every link of these chains.

6.2 - Important improvements were recently introduced in the probabilistic modeling of earthquake actions for seismic design. However further research in this area is needed.

6.3 - The large randomness inherent to the occurrence and intensity of seismic actions controls the randomness of the response and consequently the probability of reaching the different limit states. In usual conditions this justifies the use in seismic situations of probabilities of failure larger than those used in normal situations.

6.4 - To improve the procedures and prescriptions related to seismic design it is important first to clarify the requirements and performance criteria involved.

6.5 - Seismic ultimate limit design should be based on available ductility. To guarantee that the ductility can be explored, separate checkings of ductility and structural integrity have to be carried out. Non-linear behaviour should be conveniently assessed.

6.6 - The experimental and theoretical information on available ductility of different structural types and on resistance under reversed shear, torsion and combined action-effects is scarce (17). Further research in these fields is urgently needed (18).

6.7 - Detailing has a large influence on available ductility. Consequently the theories which should allow to predict the ductility of the structures should consider the detailing prescriptions.

6.8 - Modern methods of dynamic structural analysis allow satisfactorily to define required ductility. The quality of the results mainly depends on the assumptions on seismic actions and basic mechanical behaviour under repeated loading.

6.9 - The principal difficulties on the checking of serviceability limit states derive from the definition of the allowable displacements. This definition should consider the type of non-structural elements and the costs involved in their repair.

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