Rubrik: Theme IV: Experimental studies concerning reinforced, prestressed and partially prestressed concrete structures and their elements

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Experimental Studies concerning Reinforced, Prestressed and Partially Prestressed Concrete Structures and their Elements

Etudes expérimentales concernant les structures en béton armé, en béton précontraint et en béton partiellement précontraint, ainsi que leurs éléments

Experimentelle Untersuchungen über Bauwerke in Stahlbeton, in vorgespanntem und teilweise vorgespanntem Beton sowie über deren Elemente

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1 - INTRODUCTION

The purposes of this report are: (1) to review experimental studies that have been carried out regarding the behavior of reinforced, prestressed and partially prestressed concrete structures and their elements, particularly their resistance and ultimate deformability under generalized actions; (2) to summarize the main results obtained in these studies and to discuss their significance; and (3) to discuss research needed for further advancement of knowledge in this field.

At present, it is generally accepted that the logical approach to the design of a structure, is the so-called limit-state approach, or comprehensive design [1,2]. Therefore, to design a structure, one must be able to predict its mechanical behavior at each significant level of the critical combinations of all actions to which the structure may be subjected during its service life. Because it is usually impractical to consider the real behavior under the actual generalized critical actions to which the structure may be subjected during its useful life, it is common to base the structural design on idealized conceptions of mechanical behavior under simplified actions. Since approximate methods are used, it is necessary to evaluate their reliability. To do so one must: (1) analyze all possible real actions which could be exerted on the structure during its service life; (2) determine the significant parameters controlling the expected real behavior of the structure at different levels of the critical combinations of real actions; and finally, (3) compare this expected real behavior with that resulting from simplifications introduced in the proposed methods. Thus, in trying to assess the reliability of any proposed method of design one should recognize, first, that it is not possible to separate the prediction of the mechanical behavior of a structure from the actual actions that will be exerted on it during its service life; and second, that in general the real behavior can be determined only by carefully planned experimental and analytical studies.

The sources, treatment, and effects of the different types of actions which may be exerted on structures are summarized in Table 1 [3]. Most of the experimental evidence available so far regarding real mechanical behavior has been obtained under the assumption that the actions which control the design of the structure are only external forces which can be treated as statically or

quasi-statically applied loads. Furthermore, it has been assumed that these forces increase monotonically and proportionally from zero up to the values that are necessary to induce what has been termed the instantaneous collapse. In practice, of course, no building ever enjoys this type of action. It may be subjected not only to force, but to imposed deformations and to the effects of changes in the environment to which it is exposed. All these actions are, strictly speaking, dynamic rather than static in nature. Most often the sequence of the actions to which a structure may be subjected consists of unpredictable fluctuations in the magnitude, direction and/or position of each of the individual actions and the only things that may be estimated with certain accuracy are the extreme values between which each of these actions will oscillate. These types of actions have been classified in Table 1 as generalized or variable-repeated actions.

TABLE 1 - SOURCES, TREATMENT AND EFFECTS OF ACTIONS ON STRUCTURES

SOURCES	TREATMENT ACCORDING VARIATION WITH TIME		EFFECTS AND PROBLEMS
	RATE	HISTORY	
APPLIED FORCES	STATIC OR QUASI STATIC	MONOTONIC — PROPORTIONAL / NON-PROPORTIONAL	FUNCTIONAL FAILURE (Unserviceability) — STRUCTURAL FAILURE {Instantaneous / Collapse (strength)} — STATE OF DEFORMATION (stiffness) Different paths may lead to different final states of deformation.
IMPOSED DEFORMATIONS			
CHANGE IN ENVIRONMENT	DYNAMIC	GENERALIZED OR VARIABLE REPEATED	LONG ENDURANCE FATIGUE (strength) / LOW CYCLE FATIGUE (stiffness, strength) / INCREMENTAL COLLAPSE (stiffness, strength)

The phenomena associated with variable-repeated actions are classified in three categories: (1) long-endurance fatigue; (2) low-cycle fatigue; and (3) incremental collapse. If the number of important action fluctuations is very large, the dominant design consideration is that of long-endurance fatigue, and no important inelastic behavior of the structure should be permitted. Low-cycle fatigue is the phenomenon associated with cyclic repetition of alternating inelastic strain, which may lead eventually to the fracture of the structural materials. Therefore, it is clear that this type of failure occurs if the variable repeated actions are essentially reversible in character. A review of results regarding low-cycle fatigue indicates that the real danger of repeated-reversible actions is not fracture of the structural material, as has usually been described in the literature. Rather it is deterioration of the stiffness [4]. Incremental collapse is the phenomenon associated with the progressive development of excessive deflections which occur under cyclic application of different combinations of peak actions.

The two patterns in which variable-repeated actions have been idealized-- when the phenomena of incremental collapse and low-cycle fatigue have been treated separately--appear to produce similar undesirable effects on a structure, i.e., increase in deformations [4]. Therefore, in investigating the importance of real generalized actions the effects of alternating excitations cannot be isolated from the effects caused by excitation patterns leading to incremental deformation [3]. In this report, attention will be focused on the effects of earthquake ground motion on structural concrete buildings, because this excitation is perhaps the most typical kind of generalized (dynamic, variable-repeated) action that occurs in practice. Furthermore, most of the recent comprehensive programs of experimental investigation on effects of repeated forces have been conducted in the field of earthquake engineering, particularly the effects of severe earthquake ground motions on buildings.

To recognize the significance and pin down the limitations of the results obtained in the experimental studies that have been carried out and to plan rational future testing programs, it is convenient to discuss, first qualitatively, the real seismic-design problems of buildings. In the case of earthquake excitations it is usually necessary to predict the force-displacement relationship for each story with accuracy. The possibility of low-cycle fatigue as well as incremental collapse, as illustrated in Fig. 1, must be

considered. The lateral displacement at any story, [ΔH_i], can be expressed as a function of the gravity forces acting in the structure [G(t)], the three components of the ground displacement [$U_g(t)$], and the dynamic characteristics of the soil and structure, which could be represented symbolically by the period [T(t)] and the damping coefficient [$\xi(t)$]. Thus:

$$\Delta_{H_i}(t) = f[G(t), U_g(t), T(t), \xi(t)] \tag{1}$$
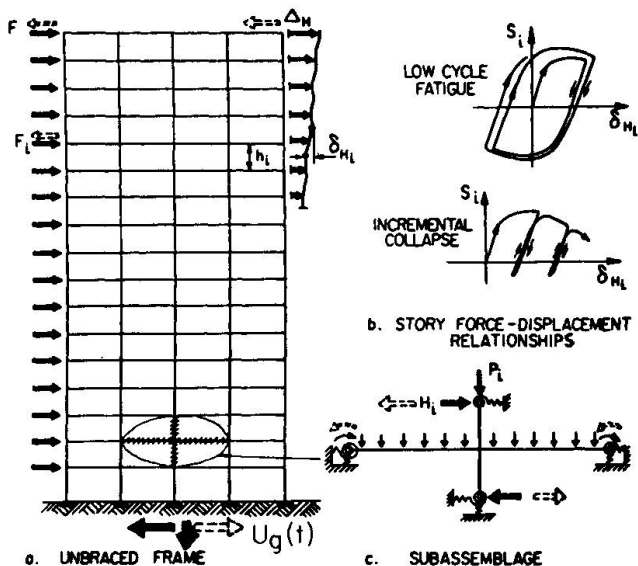

FIG. 1 - PROBLEMS IN FRAMED STRUCTURES

Analysis of the parameters involved in equation 1 clearly indicates the difficulties that arise when one tries to predict response against earthquake ground motions. The first difficulty arises from the fact that, strictly speaking, all these parameters are functions of time, although the gravity forces usually remain practically constant for the duration of an earthquake. Therefore we are dealing with a dynamic problem in which it is necessary to consider the following two important effects of the time variation of the actions and of the response; first, the effect of the inertia forces developed at the masses cannot be neglected, and second, the rate of change in intensity of the actions with time (rate of loading or straining) might be high enough to affect considerably the so-called static-mechanical characteristics of the materials on which the dynamic characteristics of the structure [T(t) and $\xi(t)$] are usually predicted.

Because the inertia forces depend not only on $U_g(t)$ but also are influenced by the behavior of the structure, [$\Delta_{H_i}(t), T(t), \xi(t)$], it is clear that this interaction between the structural response and the forces themselves brings serious difficulties not only to theoretical prediction of the response, but also to programming any rational experimental investigation. As will be discussed later, the only way to overcome these difficulties efficiently is through close integration of both analytical and experimental studies of the same problem.

Another difficulty that arises in trying to predict $\Delta_{H_i}(t)$ is that this depends on the ground motion occurring at the foundation of the building, rather than on the so-called free-field ground motion [$U_g(t)$], which is often the only one measured and reported. The actual ground motion at the foundation of the structure depends on the soil-structure interaction. This interaction affects not only the ΔH_i by the so-called rocking and sliding effects but also the overall values of T(t) and $\xi(t)$. Still, another series of difficulties arises from the fact that the behavior of actual buildings generally cannot be predicted accurately just from the characteristics of simple structural elements, because these elements interact on one another and with non-structural elements in a complex manner which depends on the detailing of their joints and connections. Furthermore, because we are interested in predicting the deformational response well into its inelastic range, we have to determine both the critical loading sequence and the history of loadings, because it is well

known that inelastic deformations are not simple-valued functions of stress, i.e. the state of strain depends upon the prior deformational history of the structure. Thus, the principle of superposition does not apply.

The above difficulties are a clear indication of the need for experimental studies of actual buildings under real actions. Thus, this report starts with a review of what has been done to date in this field. Then it covers the different approximate experimental approaches that have been used. A comprehensive review of the investigations carried out before 1966 on the effects of load repetitions in structures was presented by Saillard [5].

2 - EXPERIMENTAL STUDIES ON THE BEHAVIOR OF ACTUAL STRUCTURES

2.1 Tests Under Real and/or Simulated Generalized Actions

The need for instrumentation of actual buildings to determine their behavior under the real actions has been already pointed out by several researchers [6].

In Japan, several reinforced concrete buildings and their surroundings have been thoroughly instrumented and under observation for several years. The main objectives of these studies are: first, to determine dynamic strains and stresses as a function of the ground motion during earthquakes and explosions; second, to clarify the subsoil and superstructure system and the input ground motion to that system; and third, to use these results to improve the theory of earthquake response and the seismic design of buildings [7,8]. One good example of these instrumented buildings is the ERI (Earthquake Research Institute) main building at the University of Tokyo. This building has been already subjected to a large number of earthquakes. Although some of these earthquakes were of sufficient intensity to induce measurable deformations, none of them has been "strong" enough to induce inelastic deformations. However, some very significant results have been obtained regarding the behavior of the building under what can · be called service earthquake loads. Among these results the following can be noted:

(1) When the experimentally obtained values for strains and predominant period of the building were compared with those obtained from linear elastic analysis of the bare structure, great differences were observed. These differences were due mainly to the participation of the non-structural elements (slightly reinforced concrete and concrete block-partition walls). The contribution of these non-structural elements was determined with the aid of additional experimental and analytical studies. Including this contribution the agreement between analytical and experimental values was fairly good. Thus, this study shows the importance of the so-called non-structural elements in the initial stage of an earthquake. However, more research is required to determine what happens once these non-structural elements crack or fail.

(2) The experimental data have also made clear the importance of the different types of damping that contribute to the total effective damping [$\xi(t)$] as determined from vibration tests. The results show that a considerable part of the actual effective damping was due to radiation damping. The actual damping of the structure itself was considerably less than the usually assumed values.

The analysis and interpretation of the data obtained so far from these instrumented buildings and the additional analytical and experimental studies associated with them, have revealed some very important lessons regarding the elastic behavior of real buildings. However, practically no data has yet been obtained regarding the behavior in which this Congress is interested, that is, deformational response in the inelastic range. The chances of obtaining the response of real buildings to extreme loading conditions produced by an actual severe earthquake or windstorm have increased since recently several new buildings have been instrumented in several countries. Still, we cannot afford to wait for these extreme excitations to occur in the vicinity of these few

buildings to learn about actual inelastic behavior and the safety involved in
our present design methods. Therefore, new approaches to obtain the missing
information should be attempted.

Ideally the next approach would be testing actual instrumented structures,
under simulated extreme excitations. However, such an approach faces serious
problems, such as: (1) how to generate these extreme excitations, especially in
the case of ground motion; and (2) how to assure that these excitations will not
affect other structures in the neighborhood. While the second problem can be
solved by constructing structures in isolated places, the technical and economic
solution of the first problem is still not clear. Thus a more feasible approach
should be attempted.

In the case of earthquakes, a logical approach is to reproduce ground
motions by means of controllable shaking tables [9]. This is a method that has
been selected by most of the earthquake research institutes. Many small-size
and a few medium-size simulator facilities are already in use. To the best of
the author's knowledge, the largest facility in use is the one in Tsukuba,
Japan, which incorporates a 15 m x 15 m shaking table. Although the research
potential of these facilities is excellent, they can test only full-scale
structural elements and assemblies or small-to medium-scale models of complex
structural systems. These models are usually inadequate to investigate in
detail the actual dynamic characteristics and failure mechanism of the proto-
type. Furthermore, the reproduction of the actual ground motion (three compo-
nents of displacement) is not easy. Most of these shaking tables reproduce only
one of the three components at a time.

An extensive feasibility study of simulator facilities carried out at
Berkeley [9,10] proved that it will be feasible to construct a shaking table
which can simultaneously undergo the three translational components of ground
motion. However, the test structure could still be no larger than a 100 ft. x
100 ft. three-story reinforced concrete building.

From the above considerations, it is clear that at present testing actual
complex structures under extreme dynamic excitations, real or simulated, does
not appear feasible. Thus, the most logical approach appears to be one in which
actual structures or large-scale models are subjected to equivalent pseudo-
static forces which induce effects similar to those of real dynamic excitations.

2.2 Tests Under Equivalent Pseudo-Static Forces

In this case, the time effects are apparently eliminated. That is, inertia
forces are replaced by equivalent pseudo-static loads, and effects of the rate
of straining and $\xi(t)$ become negligible by applying these loads slowly enough.
Then for the example of Fig. 1, the lateral displacement [ΔH_i] can be expressed
as a function of the following parameters:

$$\Delta_{H_i} = f[G, H(\text{history}), K(\Delta_H), F(\Delta_H)] \qquad (2)$$

where:

    $H(\text{history})$ = lateral forces, magnitude and history

    $K(\Delta_H)$ = stiffness of the structure, which in the inelastic range
                    varies with the deformation history.

    $F(\Delta_H)$ = foundation effects which will also vary with the
                    deformation history.

It is essential that we recognize from the start the limitations of this
method. In real situations, the inertia force at each concentrated mass varies
with time, depending on the interaction between the real dynamic excitation and
the dynamic characteristic of the building. Therefore, the simulation of the
actual inertia forces by simple static forces is a very difficult problem. The
only solution is to simulate what can be considered the critical combination of
inertia forces that could be developed at a certain time. Rational selection

of this critical combination requires integrated analytical and experimental studies, because it will vary, depending on what we are interested in studying. For example, in tall buildings the critical combination of the inertia forces will depend on the story we would like to study. Furthermore, even if a rational combination of inertia forces can be selected, the problem of how to vary the magnitude of these forces still remains. It is well known that the behavior of reinforced concrete is very sensitive to the loading path [11]. The selection of the proper load sequence requires the integration of results from both analytical and experimental studies, i.e., to analyze the response of the building using mechanical models obtained from experimental results. This would require an iterative approach in which at first a simplified mechanical model is assumed to estimate a critical loading sequence. Using this loading sequence in the experiments permits improvements to the mechanical model, which then can be used in a new dynamic-response analysis. This will then lead to the selection of a new loading sequence for the test, and so on. This method has just recently become possible through newly developed, sophisticated, nonlinear dynamic-analysis computer programs and it will be used in future tests at Berkeley.

Since for earthquakes it is not easy to predict the actual critical distribution of inertia forces at different times, tests are usually carried out using a distribution of static forces recommended by codes. The magnitude of these forces is varied according to arbitrarily selected sequences, which usually are not the most critical ones. Furthermore, the effects of the vertical component of the ground motion are usually neglected in the tests.

Japanese researchers already have tested several actual structures up to failure using repeated reversal pseudo-static lateral forces. Some examples of these tests and their most significant results are discussed briefly below.

1. Tokyo Telegram and Telephone Building [5,12]. This six-story reinforced concrete framed building was constructed in 1932. In 1965 a single transverse six-story, three-bay frame was cut from the rest of the building and was tested by applying static lateral forces of equal magnitude at the roof, and fifth and third floors. These forces were increased in eight steps from 20 t. up to the maximum of 153 t. The forces at each step were applied first in one sense and then in the opposite, except for the seventh step. After each of these steps dynamic free-and forced-vibration tests were carried out. There were some discrepancies between analytical and experimental values and from the data available, it appears that the main reason for these discrepancies was that the horizontal and vertical displacements of the footings were not considered in the computations. The tested frame did not have a beam joining the footings. The behavior of the frame was very ductile and there was considerable degradation in the initial stiffness after each load reversal. The free and forced vibration tests revealed that the experimental value for the fundamental period after the application of the first reversal of lateral loading at the working load level (20 t) was about 0.9 sec. This value increased as the peak value of the applied cyclic loading was increased, and after the last cycle it was about 1.25 secs. The analytical computed values were 0.8 secs. and 1.70 respectively. The experimental values for the damping coefficient (ξ) varied from 2.5% at the working load level to a value of 6.5% after the last cycle of load reversal.

2. Old Tokyo Marine and Fire Insurance Building [13]. Although this building was made only partially of reinforced concrete, its test was very important, because it survived the 1923 Kanto earthquake with minor damages. The elasto-plastic behavior of a frame of this building was investigated up to failure by applying statically repeated-alternated loads. Changes in the dynamic characteristics as the applied deformation increased were also investigated by means of vibration tests. The frame developed high ductility as the final deflection was about 10 times as much as the deflection at elastic limit. The results of the test also show that as a consequence of alternating loads, a noticeable decrease (degradation) in the initial stiffness of the whole structure took
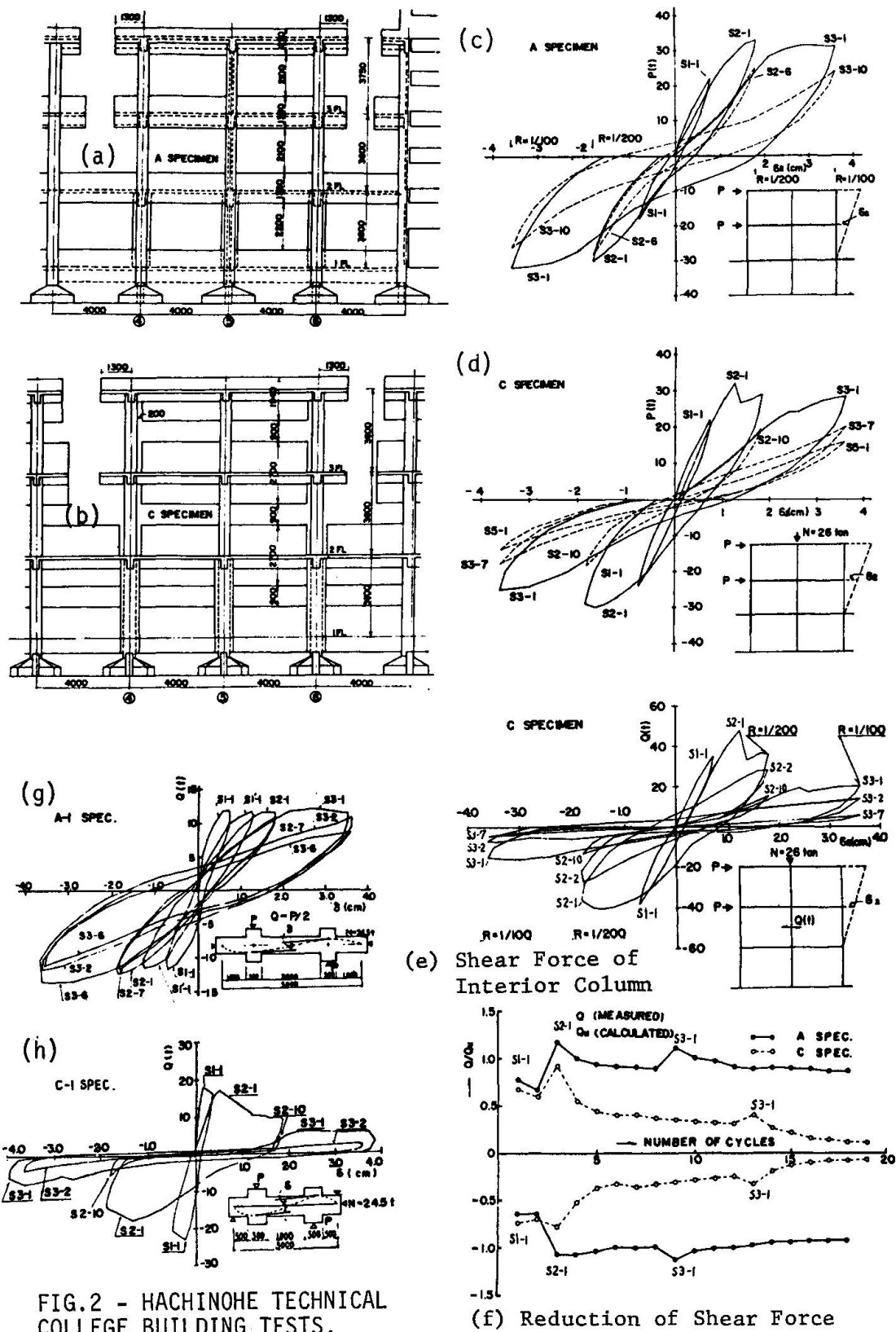
place. Regarding the effect of this reduction in stiffness on the dynamic
characteristics of the building, it is interesting to note that the fundamental
period determined from vibration tests varied as follows: (1) before the load-
ing test, i.e., corresponding to the elastic stiffness of uncracked section,
it was 0.98 sec.; (2) after loading of individual floors to determine flexibil-
ity coefficients, it was 1.29 sec. (the observed increase was due to the forma-
tion of cracks),and (3) after all the static loadings were completed, the
fundamental period was 1.98 sec. which implies a reduction from original stiff-
ness of more than 400%.

As far as damping coefficients, and their variation with increased damages,
the following results were obtained: (1) before loading the damping coefficient
was 3.6% of critical damping for the first mode, and the damping coefficients
associated with higher modes were rather small. This suggests that the damping
characteristics of this frame cannot be approximated by the conventional stiff-
ness proportional internal damping which increases in proportion to the natural
frequency; (2) damping increased to 7% for the first mode after the ultimate
load.

3. Hachinohe Technical College Building [14]. This three-story, reinforced
concrete building, suffered serious damage during the 1968 Tokachi-oki Earth-
quake. The structure consists of frames in the longitudinal direction and in
the transverse direction frames with shear walls. The longitudinal frame on
the south side (A frame in Fig. 2a) has relatively long columns with shallow
spandrel walls which leaves the column with an h_0/D ratio of 4.6 (h_0 is the
clear height of column and D is its depth). The longitudinal frame on the
north side (C frame in Fig. 2b), on the contrary, has short columns with very
deep spandrel walls giving an $h_0/D = 2.2$. It is important to note that the
spandrel walls were not considered as structural elements in the analysis car-
ried out on the design of this building.

During the 1968 Tokachi-oki Earthquake the most serious damage took place
in the longitudinal direction and in the first story. Short columns in the C
frame showed shear failure, with X-shaped cracks and extensive crushing and
spalling of concrete and buckling of bars. On the other hand the relatively
long columns in A frame showed shear and/or bending cracks at the top and bot-
tom, with just occasionally crushing of concrete and spalling of finishing
mortar. The columns of the second story had only minor cracks, and no damage
at all was observed in the third story. A series of field tests, combined with
laboratory experiments and analytical studies, were carried out as an attempt
to analyze the actual behavior of this structure during the earthquake. The
field tests consisted of subjecting isolated parts of the building to repeated
reversal of static loading conditions up to failure, as well as free and forced
vibration tests.

The specimens A and C shown in Figs. 2a and 2b consisted of two-span, two-
story frames which were cut from the second and third stories of the building.
The damaged first-story columns were strengthened. The results of the static
tests are shown in Figs. 2c and 2d in the form of load-displacement curves. The
A specimen (long columns) failed by flexural yielding. After 10 cycles of full
reversal at the maximum deflection of 3.6 cm in the second story (which corre-
sponds to a rotation angle [R] of 1/100 radians), only very minor crushing of
concrete was observed, although considerable degradation in initial stiffness,
strength, and energy dissipation was observed with the increase in the number of
cycles. (Compare first cycle S3-1 with last cycle S3-10 in Fig. 2c). The C
specimen failed in shear. As is indicated in Fig. 2b, the two exterior columns
in this specimen were made considerably more slender than the interior ones by
separating the spandrel wall from the column by slits. The shear failure that
took place at the interior second-story column started to develop under a
deflection of just 1.2 cm (R = 1/300), as is clearly indicated in Fig. 2d by the
sudden drop in strength during the cycle S2-1. This failure became quite exten-
sive at the deflection of 1.8 cm (R = 1/200). After seven cycles at the

FIG.2 - HACHINOHE TECHNICAL
COLLEGE BUILDING TESTS.

(e) Shear Force of
Interior Column

(f) Reduction of Shear Force

deflection of 3.6 cm [R = 1/100], the interior column was so damaged that the specimen was not able to carry the vertical load. The failure of the interior column is emphasized in Fig. 2e, in which the story displacement is plotted against the shear force which corresponds to the interior column alone. Note the sudden drop during cycle S2-1, as well as the considerable decrease in initial stiffness and in the area of the hysteresis loop (dissipation of energy) as the number of loading cycles increases. Comparison of the results shown in Figs. 2c, 2d, and 2e points out the effect of shear on the ultimate behavior of columns, and therefore shows the need to avoid shear failures. The reduction of column shear with the number of cycles is illustrated in Fig. 2f.

Test specimens similar to the columns of frames A and C were fabricated and tested. The loading conditions were similar to those used in the field tests and the results are shown in Figs. 2g and 2h. Again, while specimen A-1 failed in bending, specimen C-1 failed in shear. The significant contribution of these tests is that the results show clearly how an expected ductile bending mode of failure can be changed to a brittle shear mode of failure by just decreasing the slenderness ratio of columns. Similar results can be obtained using main reinforcing steel with a yielding strength higher than that called for in the design. The big difference in the inelastic behavior of similar elements under repeated reversed loads (when shear, rather than bending, starts to control the response and the mode of failure) is clearly illustrated by the curves of Fig. 2.

Analytical studies of the response of this building to several earthquake motions show that the response is very sensitive both to changes in restoring force characteristics, especially in the area of the hysteresis loops, and to the characteristics of the earthquake input [15]. This clearly shows the importance of determining accurately the actual, complete load-deformation history, which will be a function of the earthquake input. Thus to determine it by test requires the iterative method discussed previously.

Because there are few opportunities to do field tests of actual buildings up to failure, and because of the difficulties in instrumenting and loading buildings in the field (which as seen usually leads to tests on a simple isolated frame of a building),the next approach for determining the actual behavior of buildings is to carry out laboratory tests on large-scale models of actual buildings.

2.3 Test of Large-Scale Models Under Equivalent Pseudo-Static Lateral Forces

Since 1967, Japanese researchers have been carrying out static and dynamic tests on full-size apartment buildings of up to five stories and built of cast-in-place reinforced concrete, precast reinforced concrete, and precast concrete with a prestressed construction system. New and complex frame-walled structural systems have been tested. The results of these tests have been published in a series of reports [16-19]. These studies have clarified the actual seismic behavior of these complex structural systems, particularly on the interaction of their different elements. These results would have been practically impossible to predict analytically or by only separate tests of their individual structural components. In complex structures like this it is very difficult to make accurate estimation of how the different elements interact. There are no clearly defined girders and columns, and it is not easy to estimate how much of the slabs and walls contribute to the strength and stiffness of idealized girders, columns and shear walls. Only tests of this nature can give an idea of actual behavior. From results obtained in later tests and related analytical studies, it has become evident that it will be possible to build fifteen-story housing structures of either cast-in-place reinforced concrete or precast-prestressed concrete which will endure extreme earthquakes [16].

Figure 3 illustrates results obtained in the tests of a specimen which consisted of full-size structural elements corresponding to the lower four-stories of a real eight-storied precast reinforced concrete apartment house assembled with a post-tensioning method [19].
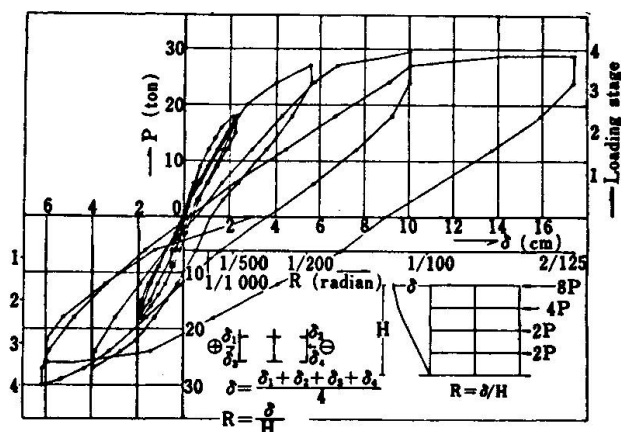
FIG.3 - LOAD-DEFLECTION CURVES [19].

Regarding the effects of repeated reversal loadings, the most significant conclusions obtained from these full-size structural tests can be summarized as follows:

(1) There is a degradation in initial stiffness and in the energy-dissipation capacity when the structure is subjected to a new reversal loading cycle after the deformation has been increased in the previous cycle.

(2) The fundamental period for walled structures at working load level varied from 0.1 sec. to 0.15 sec. In the inelastic range, this period became larger and larger as the structure was subjected to larger numbers of loading cycles with increasing peak deformations. After the cycle in which the maximum static lateral forces were applied, the period varied from 0.17 sec. to 0.47 sec.

(3) The damping coefficient, which in the elastic region ranged from 1.4% to 2.2% under small amplitude vibration, increased to values which ranged from 4.7% to 8.7% after the loading cycle in which the maximum static external force was applied.

(4) When the specimens tested have walls whose shear strength and rigidity were not uniformly distributed, the weaker walls cracked under very small lateral forces and failed severely in shear. In these cases, the load capacities were smaller than those estimated and their strengths decreased rapidly with the increase in number of cycles of reversal loading. On the other hand, the specimens that were properly designed and detailed have sufficient strength, and this strength was not affected by the repeated reversal of forces, and they have sufficient ductility and energy dissipation capacities to withstand the effects of the generalized forces which could be developed in extreme earthquakes.
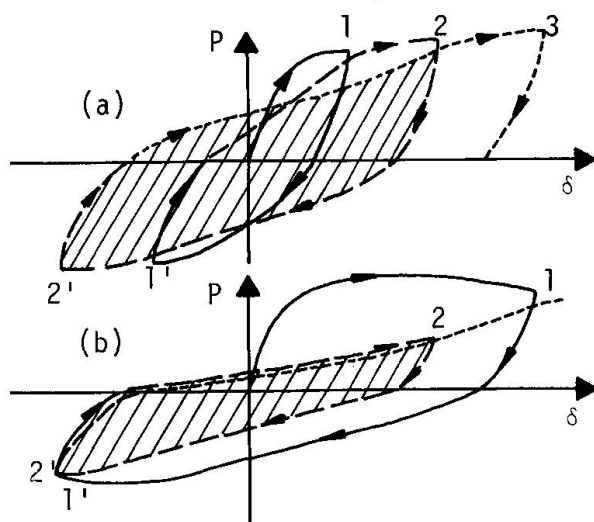

FIG.4 - EFFECT OF LOADING HISTORY
ON ENERGY DISSIPATION.

Regarding the validity of these results, it is important to recognize that the distribution and sequence of the applied forces which have been used might not represent the critical patterns which can be induced in extreme earthquakes. The hysteresis loop for fixed peak deformation depends on the previous history of loading. This is illustrated in Figs. 4a and 4b. If the fixed peak deformations have never been exceeded before,(Fig. 4a), the peak resistance, initial stiffness and energy dissipation are larger than in the case where the fixed peak deformations have been exceeded in a previous cycle (Fig. 4b). From these observations, it appears that the application of repeated reversal loading cycles in which the peak values of the load and/or deformation is increased gradually (which is the usual method of testing) might not be a "conservative" way of testing. The

structure might show considerably less energy dissipation capacity, and even less maximum strength, if it is loaded near or up to its ultimate resistance (deformation) during the first cycle [23].

3 - STUDIES ON BEHAVIOR OF REINFORCED CONCRETE STRUCTURAL ELEMENTS AND SUBASSEMBLAGES

Although there is no doubt that it would be ideal to test a real building under the actual loading conditions to which it may be subjected to during its service life, such tests are not usually feasible due to economic considerations. Very significant experimental work has been carried out on small and medium scale models of structures under generalized forces [20,21,22]. Usually the main objective of this work was to find out the behavior of a specific structural system rather than to study the general behavior of reinforced concrete structures under this type of excitation. Thus, the next logical approach is to try to predict the response of the complete building or its structural system from results obtained in studies carried out on its structural elements. This has been the approach followed by most investigators. In this approach the first question to be answered is: Can the response of the whole structure be predicted from knowledge of the behavior of its components? In discussing this problem, Newmark and Hall have made the following statement: "The strength of the combined system, the damping in it, and the mode of failure can in some cases be inferred from the properties of the individual elements: however, these members interact on one another in a complex way and in different ways for different types and directions of loading, and the interaction is a problem which must be taken into account in detail much more accurately than has been the case in the past if adequate lateral resistance to dynamic forces is to be achieved." [6]. Test results presented in section 2 of this report confirm the importance of interaction. Assuming that proper consideration of this interaction gives a positive answer to the above question, then the next question is: "What are the basic components or structural elements whose behavior should be studied?" The answer to this second question, of course, depends on the structural system. Even if we reduce our discussion solely to reinforced concrete buildings, there are several structural systems that can be used and have been. However, for the purpose of this review only two systems will be considered: (1) Ductile Moment-Resisting Space Frames, and (2) Shear Walls.

FIG. 5 - SUBASSEMBLAGES

It is believed that to predict the response of a building under generalized loading conditions accurately, it is necessary to be able to predict at least the displacement of each story. Therefore, it is clear that it is necessary to know the behavior of subassemblages such as those indicated in Figs. 1 and 5. Reasons for selecting these subassemblages can be found in detail in references 3, 23, and 24. To facilitate interpretation and evaluation of results obtained in the studies of behavior of these subassemblages, it is convenient to review first the behavior of their elements and to separate the case of reinforced concrete elements from prestressed concrete elements.

3.1   Elements of Ductile Moment-Resisting Frames

Results obtained in a series of tests on reinforced concrete frames that have been carried out at the University of California, Berkeley, since 1959 [28], as well as detailed analyses of results obtained by other investigators,

[5,6,25-27] indicate that under variable-repeated actions, the real problem in reinforced concrete structures properly designed in accordance with current requirements appears to be one of degradation of stiffness and energy dissipation rather than strength. Then the question that arises is: "What is the reason for this observed degradation?" This degradation should be a direct consequence of localized inelastic deformation that occurs at certain over-stressed regions along the members of the structure. These regions are illustrated in Fig. 6 for just one floor, and they will be denoted from now on as "critical regions".
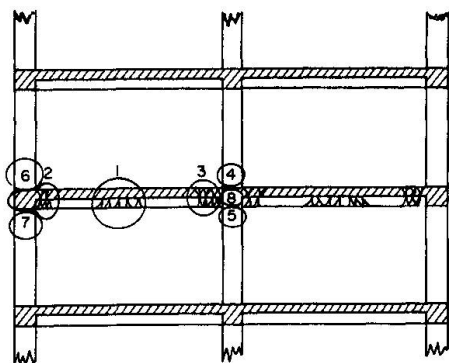

FIG. 6 - CRITICAL REGIONS

In reviewing the behavior of reinforced concrete regions, it is convenient to classify them according to their controlling state of stress as follows: (1) critical regions where inelastic behavior is controlled by bending (flexural critical regions); (2) critical regions whose inelastic behavior is controlled by the high shear existing at the region, rather than bending; and (3) critical regions whose inelastic behavior is controlled by high axial and shear forces, rather than bending.

1. Flexural Critical Regions. Many researchers have investigated experimentally the behavior of these critical regions under repeated reversed loading [23,26,27,30-32]. From the analysis and evaluation of the results obtained in these tests the following observations can be made:

(1) Properly designed and adequately reinforced concrete sections when subjected to generalized (repeated reversal) bending moments can develop ductility factors and offer energy absorption and energy dissipation capacities which exceed the values that might be required at these regions in case of a major earthquake. This is clearly illustrated by the results presented in Fig. 7.

FIG. 7 - MOMENT-CURVATURE RELATIONSHIP, BEAM 4[23]

(2) No significant stiffness degradation has been observed at the critical regions of the members tested, when these regions are subjected to repeated moments acting in the same sense and having peak values at service level.
(3) The behavior of the critical regions of the double reinforced-concrete members tested has been affected by repeated reversal of moments, when compared with the behavior expected under monotonically increasing moments, as follows: (a) The instantaneous stiffness is reduced after each reversal of moment in which the value of the peak moment and/or the deformation is increased beyond its previous value; (b) The maximum and ultimate strengths are increased if the peak of the moment and/or deformation of each successive reversal loading cycle is increased gradually; (c) Larger curvature and deflection ductility factors are obtained when the peak values of the moment reversals are increased gradually; (d) The energy absorption and energy dissipation capacities under reversal moments are higher than those expected under

monotonically increasing moment. Due to the degradation of the instantaneous stiffness and the effect of strain hardening of the steel, the prediction of these capacities cannot be based on a linear elastic-perfectly plastic model.

(4) The behavior under reversal moments as the deformations imposed on the critical regions increase beyond the first yielding is controlled by the mechanical characteristics of the steel. This is a consequence of bond deterioration* that occurs between main cracks as the reversals of moment increase in number and in magnitude of peak value. To predict such behavior will require the knowledge of the variation of the stress-strain relationship of the steel reinforcement under a strain history similar to that to which it is subjected in the critical region of the member. The Bauschinger effect is one of the controlling factors in the observed degradation of the stiffness. A method for predicting this degradation has been presented in reference 25.

(5) Final failure usually has been due to buckling of the main reinforcement. The main factors controlling this buckling are: (a) concrete cover; (b) the spacing, size and detailing of hoops; and (c) strain history of the steel bar. Thus, application of results obtained in the test of small-size elements to larger sized members should be done with caution, especially if some of the above parameters are not simulated properly.

(6) The degradations in stiffness and in energy absorption and energy dissipation capacities with repeated reversal loading cycles of increased magnitude of deformation is not so severe as those observed in case of frames and buildings.

(7) There is not a clear correlation among the strain, curvature and deflection ductility factors, except after severe cycles of reversals. This is illustrated in Fig. 8.

FIG 8

VARIATION OF DUCTILITY
FACTORS FOR STRAIN,
CURVATURE AND DEFLECTION
FOR BEAM No. 4 [23]

In reinforced-concrete structures, knowing accurate values for the required ductility at critical regions is particularly important, as available rotation capacity is sensitive to type, amount and detailing of reinforcement, as well as on the history of deformation. Depending on the type of structural system, the relative stiffness of members, and the ways that these members and their connections are detailed, as well as on the history of deformations, the ductility requirements for the local deformations that take place at the critical regions may considerably exceed the ductility requirements for the lateral displacement [23,29]. Therefore, requirement of a lateral-displacement ductility factor, as is usually done at present, although necessary, is not by itself

*The influence of load history on the deterioration of bond has been studied experimentally at Berkeley [33] and at Rice University [34].

sufficient to prevent failure under the severe generalized excitations that occur during an extreme earthquake.

   2. **Flexural Critical Regions With High Shears.** The data available on behavior of this type of critical selection subjected to repeated reversal of loading is scarce [31,35-37]. Evaluation of the available test results indicate:

   (1) Reduction in the shear span or increase in the shear forces at the critical region reduced the number of cycles to failure.

   (2) As soon as the average shear stress at the critical region reaches values larger than about $3\sqrt{f_c'}$ (psi), the degradations in stiffness with reversal of load and in energy absorption and energy dissipation capacities with increase in the number of cycles become considerably larger than those for flexural critical regions with very low shear stresses. Also, degradation in strength occurs as the number of similar loading cycles increases. Failure is initiated by shear. Mechanism of failures have been suggested in references 31 and 35.

   (3) The ability of critical regions to maintain load and energy dissipation capacity (hysteresis loop stability) is significantly improved by reducing hoop spacing. For cases where shear stresses reach values on the order of $5\sqrt{f_c'}$ (psi) or larger and where web reinforcement has been designed according to present code recommendations, the degradations in strength, stiffness, and in energy absorption and energy dissipation increased considerably with an increase in the number of cycles and the magnitude of deformations. To improve these mechanical characteristics it is necessary to: (1) neglect any contribution of the concrete in resisting shear stress; and (2) design web reinforcement considering the actual bending capacity of regions. That is, this capacity should be based on actual strength of reinforcing steel, including strain hardening, and not just on minimum specified yielding strength.

   (4) To avoid excessive damage, and therefore degradation, when a region can be subjected to severe repeated loading reversal, the shear force which could be developed at the critical sections should be limited in magnitude to a value considerably lower than that recommended by ACI Building Code, which is $10\sqrt{f_c'}$ (psi). The larger degradations observed in this type of critical region cannot be predicted by consideration of bond deterioration and Bauschinger effect alone as in the case of pure flexure. Several significant additional factors have been already suggested in references 25, 31, and 35.

   It should be noted that if the behavior of this type of critical region is described directly by a measured load-displacement relationship, rather than by its moment-curvature relationship, the previously mentioned degradations are considerably larger because the contribution of shear deformations is not detected just by measuring rotation at the critical region [25].

   3. **Flexural Critical Regions With High Axial and Shear Forces.** It is well-known that the presence of axial forces not only changes the moment capacity of reinforced-concrete sections, but also considerably affects the ductility [6]. As a consequence of the considerable damage that occurred in reinforced-concrete columns during the 1968 Tokachi-Oki earthquake, Japanese investigators have carried out numerous experimental and analytical studies on flexural members subjected to significant axial and shear forces [14,38-43]. In all of these experimental works, compressive axial and shear forces were used. To the best of the author's knowledge, no experiment has been carried out to study the effect of generalized forces in columns that can be subjected to tension axial forces. This is unfortunate 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.

   Test results show that as the axial stress is increased, the reduction in strength and in deformational energy absorption and energy dissipation capacities increases with the number of cycles of repeated reversal   loadings. However,

for columns with shear span ratios larger than 2, by increasing the amount of web reinforcement, specially by decreasing the spacing of hoops, the reduction is limited, and it is possible to have very ductile column behavior even under high P/P_y ratio and large number of repeated alternating cyclic of bending moments. This is clearly illustrated in the results shown in Fig. 9, which were obtained in specimens having a shear span ratio of 2.14. All these specimens were reinforced with a total axial reinforcement ratio of 2.78% of SD 35 steel and subjected to a constant compression force equal to 1/3 of the compressive concrete strength of the section [360 Kg/cm^2 x (35 x 35)cm^2].

Specimen	Transverse Reinforcement			
	Type		Ratio (%)	Spacing (mm)
$A\text{-}P_w0.6$	Tied Column	350	0.61	120
$A\text{-}P_w0.9$			0.91	80
$A\text{-}P_w1.2$		40 90 90 40	1.22	60
$S\text{-}P_w0.6$	Spiral Column		0.61	60
$S\text{-}P_w0.9$		270	0.91	40
$S\text{-}P_w1.2$		SPIRAL	1.22	30
$J\text{-}P_w0.6$	Hooped Column		0.61	60
$J\text{-}P_w0.9$			0.91	40
$J\text{-}P_w1.2$			1.22	30

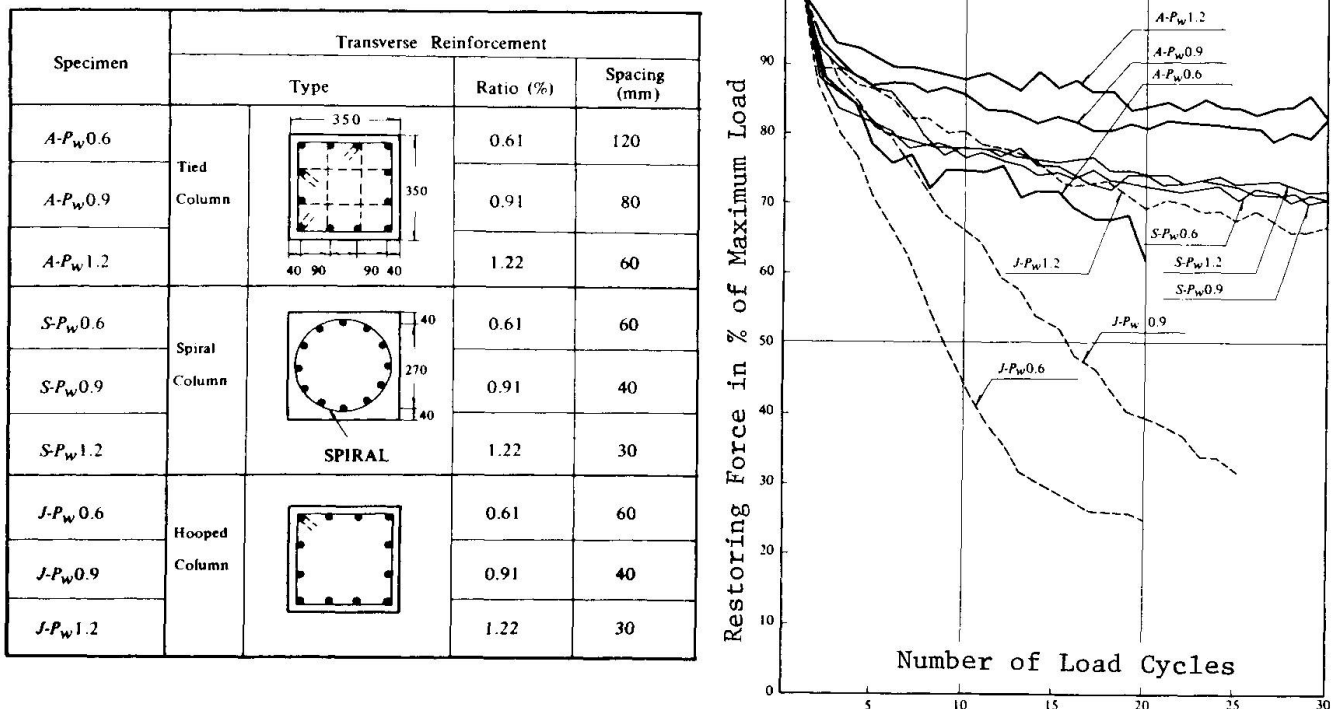FIG. 9 - REDUCTION OF RESTORING FORCE AT R = 1/100 RAD. UNDER ALTERNATING REPEATED DISTORTIONS [41]

Very short columns, in which shear forces predominate, become brittle. The results presented in Fig. 2 illustrated the effect of converting a bending failure into a shear failure by shortening the column. Studies carried out by Yamada [40] on very short columns (shear span ratio = 1.2) subjected to constant high axial forces and repeated alternating transverse loads show that when the web reinforcement ratio (P_w) is less than 0.44%, the columns show an explosive brittle failure immediately after diagonal cracking. On the other hand, columns with P_w equal or larger than 0.88% show sufficient ductility after diagonal cracking. Yamada concludes that web reinforcement ratios necessary to prevent a shear explosion failure and to give sufficient ductility under the effects of severe ground motion are as shown by tests to be larger than 1%. This requires very closely spaced hoops.

In summary, although there is a large amount of test data available the different results have not yet been compared and interpreted to deduce general rules that will permit proper design of these members against extreme generalized excitations. Furthermore, all the available data have been obtained under excitations which produce only compressive axial forces and 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, since during extreme

earthquakes tension forces can be developed due to overturning moments and the vertical component of acceleration, it is of paramount importance to carry out studies on columns in which the axial force is varied from compression to tension, while the columns are subjected to reversals of moment and shear.

3.2 Beam-Column Subassemblages

The experimental studies carried out on simple beams and columns, particularly on their critical regions, failed to show the accentuated degradation in stiffness and energy capacities that has been observed in the tests carried out in frames and actual structures. The explanation for this discrepancy has been obtained by analyzing tests on beam-column subassemblages. The bulk of the available experimental evidence to date has been provided by the tests conducted by: (1) The Portland Cement Association (PCA) [44-46]; (2) University of Canterbury Tests [47-50]; (3) Japanese investigators [51-56]; and (4) Townsend [57].

1. PCA Tests [44-46]. An extensive investigation on beam column joints for seismic resistant reinforced concrete beams concluded that well-detailed joints can resist severe earthquakes without loss in strength.

2. University of Canterbury Tests [47-50]. Figure 8 indicates that for small-displacement ductility values, there is no definite relation between the so called "strain, average curvature and displacement ductilities", and that for a specified displacement ductility the required curvature ductility at the most critical region can be considerably larger. Analyses by Park [29] have shown that in frames with beam sidesway mechanisms, the curvature ductility at the critical region may be at least 15, while the displacement-ductility factor is only about four as usually assumed in present seismic codes. Because of this and the fact that in the PCA tests the critical regions were loaded to a maximum curvature ductility of only approximately five in each direction, these tests have to be viewed in the light of the rather moderate inelastic deformations imposed [47]. Thus, Park and associates at the University of Canterbury have been carrying out a test program including not only high levels of cyclic loading with curvature ductility greater than ten, but also other parameters such as the beam-reinforcement anchorage details and the type and content of transverse reinforcement in the joint.

Results obtained in the test of 6 specimens have been reported [47,48]. Each specimen consisted of a column with a beam framing into one side at midheight, representing full-size assemblies of a reinforced-concrete external-beam-columns joint of a frame of a two-story building. In all six specimens, failure occurred in the joint regions rather than in the adjoining members due to the large joint shears. The low axially loaded beam-column joint tested showed that the joint hooping required to satisfy the minimum requirements of the ACI 318-71 building code was insufficient. The hoops near the top and bottom of the main steel were not effective as joint shear reinforcement. This suggests that the amount of joint hooping required to prevent yielding of the joint hoops is much greater than that required to take all the joint shear. Yielding of the joint hoops allowed the joint concrete to break up under successive cyclic loading.
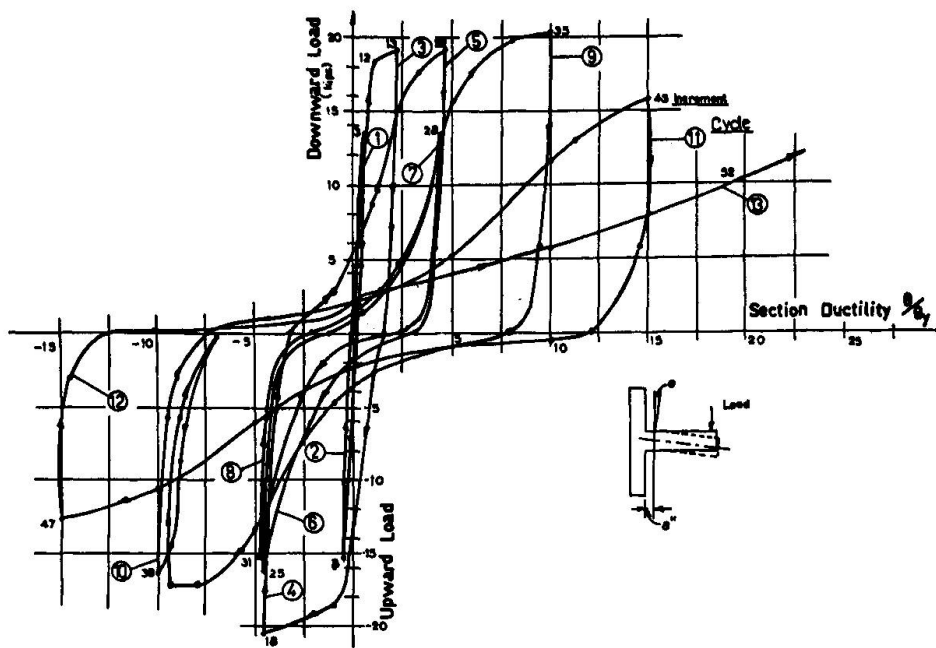
FIG. 10 - LOAD-SECTION DUCTILITY RELATIONSHIP FOR
          UNIT 6 [48]

Figure 10 illus-
trates the performance
of unit 6 which was
superior to the perfor-
mance of the other units.
Unit 6 was the only
specimen tested in which
three-quarters of the
beam's yield load could
be obtained in both
cycles 7 and 8. In
cycle 9 the load carried
by the beam was 11%
greater than the theoreti-
cal ultimate. The strain
reached in the top beam
bars in this cycle was
sufficient to produce
strain hardening. It is
interesting to note the
considerable degradation
in initial stiffness at
the reversal of the load-
ing. Results of tests

which were conducted on a new series of seven specimens have just been reported
[49,50].

3. **Japanese Tests [51-55]**. Umemura and Ikeda [51] and Koreishi [53]
reported tests of full-size assemblies of interior beam-column connections.
Higashi and Ohwada [54] have tested seventeen specimens representing, in about
1/3 scale, interior beam-column connection subassemblages of actual reinforced-
concrete frames. Several arrangements for anchorage of the main longitudinal
reinforcement of the beams within the panel zone were tested. While in some
specimens these bars were directly anchored in the panel zone, in others the bars
were continued through the panel zone. In four specimens transverse beams framing
into the panel zone at 90° were used. The main conclusions obtained from the
test results can be summarized as follows:

FIG. 11 - LOAD-DISPLACEMENT CURVES [55]

(1) Increase in the amount of
properly detailed shear reinforce-
ment in the panel zone improves
strength, stiffness and energy
absorption and energy dissipation
capacities. However, the most
effective way to improve these
mechanical characteristics of sub-
assemblages is to confine the
concrete in the panel zone with
beams framing into the panel zone.
(2) The load-displacement
curve for the specimen with beams
framing into the connection was
of "spindle shape", as illustrated
in Fig. 11a.
(3) The load-displacement
relationship for the specimen in
which the main reinforcing plain
bars of the beam were passed
through the connections was of

"pinched shape", as illustrated in Fig. 11b. In these cases, the considerable deterioration in initial stiffness with increased number of cycles was due to slippage of the beam reinforcement along the panel zone. No decrease in strength was observed with increase in the number of cycles.

(4) The load-displacement curves for the specimens without intersecting beams and in which the main-beam reinforcement was anchored in the connection was of "spindle-pinched combination shape", as illustrated in Figs. 11c and 11d. All these specimens failed due to shear cracks in the connection panels. The strength decreased with increased peak displacement and with increase in the number of loading cycles.

(5) When first yielding occurred in the columns by increasing the axial force acting in the columns (in the range below their balance point) more and wider shear cracks developed in the panel zone and the final failure mode changed from flexural column failure to a shear failure in the panel zone.

4. Townsend Tests [56]. Twenty-two half-sized reinforced concrete beam-column connections were tested. The specimens represented exterior beam-column joints of a multistory frame. The three most significant parameters investigated in these tests were: (1) column loadings; (2) deformation level; and (3) number of cycles of inelastic loading. Townsend has made a significant contribution by carrying out a thorough analysis and interpretation of the data. The hysteresis-loop data were subjected to multiple linear regression analysis. Polynomial expressions are suggested for predicting the loop, and these expressions take into consideration the observed degradation with increased number of cycles. Expression for estimating the energy dissipated is also suggested and a bond failure criterion is presented.

To conclude the review of the tests carried out on beam-column connection subassemblages, the author would like to make the following observations regarding:

(1) Types of specimen: Subassemblages like those indicated in Fig. 5, where part of the floor slab is also reproduced and where gravity forces are applied through this slab, should be tested. Reasons for this are given in reference 3.

(2) Method of Testing: It is necessary to select a definite loading arrangement and sequence of loading as standard for all the tests in order to be able to compare results directly. The author has found that the loading arrangement now in use in Berkeley [24] permits to simulate very well the main effects of gravity forces, earthquakes and/or wind storms. The selection of the proper loading sequence can be obtained only by integrating analytical and experimental studies. At present the usual sequence of loading is that of increasing the peak value of the load and/or deformation gradually. 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, such loading sequence will give upper bounds for strength and energy absorbed and dissipated. If one desires to obtain a lower bound, it is better to use a sequence starting with large peak load and deformation cycles. On the other hand, if what controls the behavior is a weak panel zone, the gradually increasing loading sequence will give a lower conservative bound. Another important consideration is the selection of the magnitude of the peak deformations in each cycle and the number of cycles to which a specimen should be subjected. Again, this question can be answered only by integrating analytical and experimental studies. If the subassemblage to be tested belongs to a very short and rigid building, it is logical to expect a large number of reversals during any given earthquake. On the other hand, if the subassemblage belongs to a very tall and slender building, the number of reversals during same earthquake will be considerably smaller. Furthermore, if the weaker elements are the beams, the number of full reversals can be considerably smaller than in the case where the weak elements are the columns and/or panel zones [57]. The magnitude of the

peak deformation to which the specimen should be subjected depends on the type of construction as well as the type of earthquake. Again, only integrated analytical and experimental studies can give correct answers.

(3) Overall behavior: From analysis of the results available it is clear that the degradation in stiffness observed with reversal of loading is considerably larger than that obtained for critical regions under pure flexure or bending and low shear forces. An explanation of this larger degradation has been offered in references 25, 49, and 56 for the case of exterior beam-column connections. The major factors contributing to this degradation appear to be diagonal cracking in the joint, crushing of the concrete around the curved portion of the anchorage of the beam-reinforcing bars and grinding of the concrete in these regions and along the diagonal cracking, which increases with the number of cycles. However, no reliable method exists at present to predict quantitatively the effect of this cracking, crushing, grinding and crumbling of the concrete in the joint. Thus, there is an urgent need for research in the behavior of panel zones under repeated reversal cycles. Behavior of interior beam-column connections also should be more thoroughly investigated than has been up-to-date.

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

3.3 Shear Walls

In view of its high resistance to lateral forces, the reinforced-concrete shear wall is regarded as a useful device for the design of buildings against wind and/or earthquake excitations. Different types of shear walls have been used in building construction. Some have already been tested against the effects of repeated forces such as might develop during an extreme earthquake.

1. Monolithic Shear Walls. Japanese investigators have carried out extensive research on the behavior of this type of wall under lateral forces [58-61]. Analysis of the results available shows that the behavior of the shear wall after first cracking, which can occur for a story drift index of about 2.5 x 10^{-4} radians, depends upon the kind of boundary frame that surrounds the shear wall, as well as the way that the wall is reinforced. If the boundary frame is strong and rigid enough and the amount of the reinforcement is sufficiently high, the first diagonal crack will not open very much and cracks will start to occur all over the panel wall. The shear wall can carry the increasing shear force without danger of a sudden failure of the wall and boundary frame. For this type of balanced design, the shear wall can absorb and dissipate considerable energy. However, it is not easy to achieve a balanced shear wall-frame-design. There are still many problems concerning the shearing resistance of such dual systems under repeated alternating load cycles. Methods and equations have been suggested to obtain a load-deflection curve for the aseismic design of shear walls [61,62] However, the actual shape of the envelope curve, which gives the relation between shear force and deformed angle, and the actual shape of the hysteresis

loops, are considerably influenced by the actual history of loading, height-to-width ratio, the amount and distribution of reinforcement, the presence of openings, etc. Thus, comprehensive studies of these parameters are urgently needed.
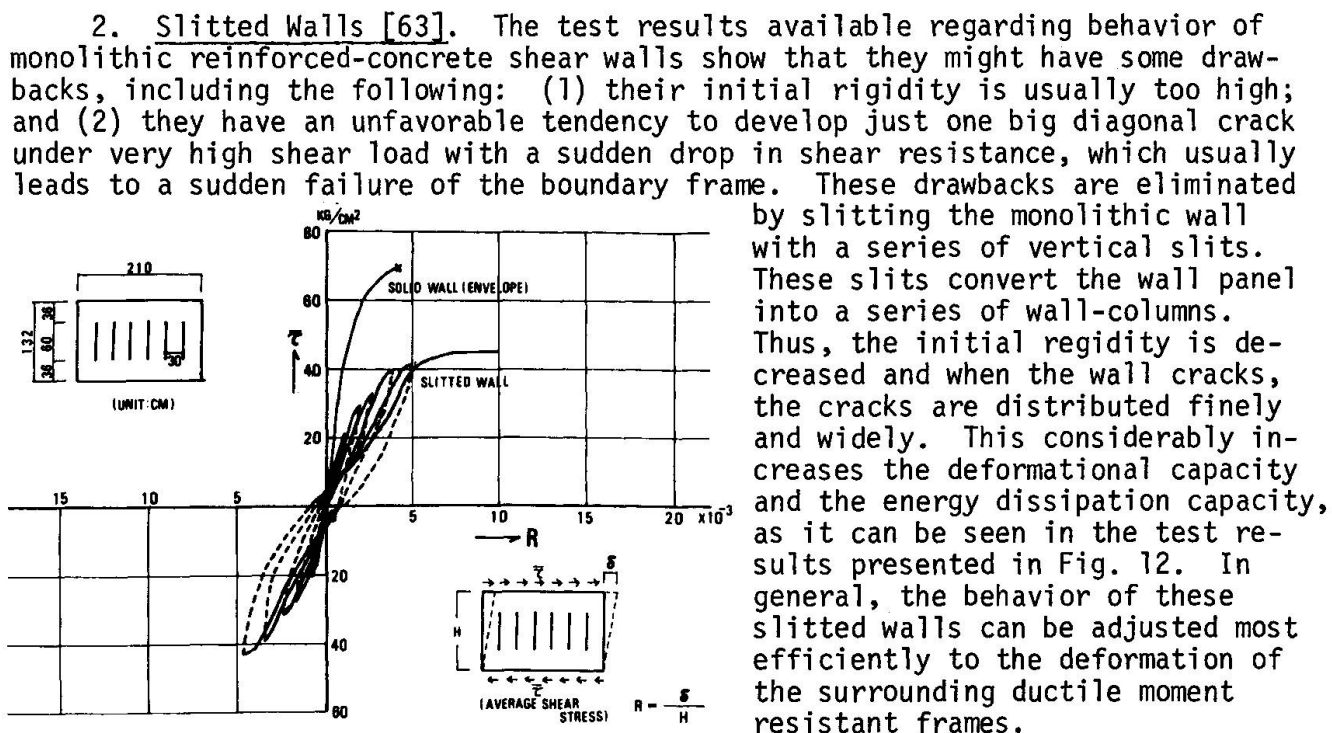
2. Slitted Walls [63]. The test results available regarding behavior of monolithic reinforced-concrete shear walls show that they might have some drawbacks, including the following: (1) their initial rigidity is usually too high; and (2) they have an unfavorable tendency to develop just one big diagonal crack under very high shear load with a sudden drop in shear resistance, which usually leads to a sudden failure of the boundary frame. These drawbacks are eliminated by slitting the monolithic wall with a series of vertical slits. These slits convert the wall panel into a series of wall-columns. Thus, the initial regidity is decreased and when the wall cracks, the cracks are distributed finely and widely. This considerably increases the deformational capacity and the energy dissipation capacity, as it can be seen in the test results presented in Fig. 12. In general, the behavior of these slitted walls can be adjusted most efficiently to the deformation of the surrounding ductile moment resistant frames.

FIG.12-SHEAR WALLS,LOAD-DEFORMATION CURVES.

Experimental and analytical works on this type of wall are needed to find out how the behavior of these walls is affected by variation in one or more of the following parameters: (1) slit length and their spacing in relation with width-height ratio of the wall; (2) the amount of reinforcement and its distribution; (3) thickness; (4) concrete strength; etc.

4. PRESTRESSED AND PARTIALLY PRESTRESSED CONCRETE STRUCTURES AND THEIR ELEMENTS

Prestressed concrete is seldom used in primary resistant structures against repeated loading conditions as severe as those expected to occur in case of major earthquakes. The principal reason for this has been the shortage of experimental evidence of the behavior of prestressed-concrete members under such loading conditions. Arya [64], in reviewing the use of prestressed concrete in seismically active areas, has discussed some questions which are raised about the use of this material in structures that should resist severe cycles of repeated reversed load.

A recent investigation carried out by Blakeley includes an excellent, comprehensive, historical review of the seismic resistance of prestressed concrete, with a complete bibliography [65,66]. In this review, he summarizes the behavior of prestressed-concrete structures during recent major earthquakes, as well as the different tests that have been carried out on prestressed concrete elements under reversed cycles of severe loadings. The main conclusions from Blakeley's review are as follows:

(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.

From the results of testing prestressed concrete members the percentage
of critical viscous damping for members is less than 2% prior to cracking, but
between 3 and 6% subsequently. In monotonic loading tests large ductilities
have been obtained.

(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 dissipation is a drawback in
seismic design. However, little is known of the energy-dissipation capacity of
prestressed-concrete members under high-intensity cyclic loading. Most tests
under cyclic loadings have been for relatively low loading intensities. The
energy dissipation would be greater for partially prestressed members once the
mild steel yields, but the joints of such members present particular diffi-
culties for precast construction.

(3) Research is needed in the following areas: damping tests of prestressed-
concrete structures; high intensity cyclic loading tests of prestressed concrete
members and subassemblages including different joint details; and non-linear
seismic analyses of prestressed concrete structures.

4.1 Blakeley's Tests [65,66]

To investigate the behavior of prestressed members under high-intensity
cyclic loading, including the behavior subsequent to crushing of the cover
concrete, Blakeley conducted a series of four tests on full-size, precast, pre-
stressed-concrete beam-column assemblies. The results included the moment-
rotation and load-displacement characteristics of the members, which indicate
their respective ductilities and energy dissipation capacities. Curvature
distribution plots, deflection profiles and steel strains in the stirrups are
also discussed.

From the analysis of the results, Blakeley concluded the following:
(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 satisfies only normal prestressed-concrete code require-
ments 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-inten-
sity load reversal; and (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 consequent diffi-
culty of repair back to a fully prestressed condition.

Attempts should be made to test beam-column subassemblages under the combined
effects of gravity and lateral forces, where the degree of prestress in the ele-
ments should be varied from "zero" up to "full prestress" in order to determine
the optimum degree of partial prestressing against the effect of severe repeated
reversal loadings.

5. EFFECTS OF RATE OF LOADING

For many years it has been recognized that the behavior of materials under
dynamic conditions is vastly different from their behavior under static con-
ditions, especially in three respects [67-70]. First, the normal static stress-
strain-relationship is altered, permitting different deformation energy absorp-
tion and energy dissipation. In general, the mechanical characteristics tend to
improve with increasing rate of load application. Second, the dynamic loading
may alter the mode of failure. In general, it enhances brittle failure and such
conditions as severe restraint, residual stresses, discontinuous flaws, thickness

of materials and joints, and transfer of stresses from one material to another in composite materials must be studied in their interaction and influence on cracking tendency. Third, dynamic loading can cause failure by fatigue, being the low-cycle fatigue of special interest to this symposium.

In spite of the above differences, at present most of the civil engineering structure designs are based on results obtained in quasi-static tests. Therefore, it is not surprising that questions continually arise concerning the effects of the actual strain rate that is induced as a consequence of the dynamic excitations to which the structure is subjected. Because the mechanical characteristics of materials, especially the strength, tend in general to improve with the increased rate of strain, it appears conservative to use pseudo-static test results in predicting behavior of structures under actual dynamic excitations. The author has certain reservations regarding the general acceptance of such approach and especially its application to concrete structures. The main reason for these reservations is that most of the reliable data on the effect of rate of straining in civil engineering structures have been limited to tensile tests of metal bars and compressive tests of concrete cylinders, or bending tests. Very little is known about the effect of rate of straining in case of flexural members whose behavior may be controlled by the presence of high shear and axial forces. What may be conservative for the flexural resistance might not be so for shear resistance, especially when the expected increase in flexural resistance might lead to a change in mode of failure from the ductile flexural one to a brittle shear one. Furthermore, very little information is available regarding the effect of rate of straining on the transfer of stresses between steel and concrete, and its influence on the anchorage of steel in concrete. In general, it can be stated that comprehensive data on the behavior of actual reinforced-concrete members under low-cycle fatigue loading at fast strain rates have still to be obtained.

From an analysis of the parameters affecting the dynamic response of structures (eq. 1) and from the above discussion, it is clear that reliable data should be obtained regarding the influence of rate of straining on: (1) the stress-strain relationship of reinforcing steel and concrete, including tension, compression and shear; (2) the internal damping of these materials; (3) the stress transfer, or bond, between these materials; and (4) the low-cyclic fatigue characteristics of each of the materials and of their composite action in different structural elements and assemblies, as well as how the detailing of reinforcing can affect this behavior, including different degrees of confinement, buckling of main reinforcing bars, different types of anchorage, etc. A brief summary of test results available on the effect of rate of straining follows.

5.1 Reinforcing Steel [67-70]

As the rate of strain increases, the following effects take place: (1) the yield stress increases to some dynamic value; (2) the modulus of elasticity increases very slightly; (3) the yield point strain increases; (4) the strain at which strain hardening begins also increases; (5) the ultimate strength increases slightly; and (6) the internal damping appears to increase. Of these effects, the increase in yield stress is most important.

5.2 Concrete [68-78]

As the rate of strain increases the following effects take place in the compressive stress-strain relationship of concrete: (1) the initial modulus of elasticity increases; (2) the maximum compressive strength increases; (3) the strain at which maximum strength is reached remains practically constant; and (4) the internal damping increases. Of these effects, the increase in the compressive strength is the most important. Very little data exist on confined concrete, but its mechanical characteristics should be expected to improve. Little data exist also regarding the shear characteristics of concrete. Since

concrete is weaker in tension than in shear, data should be obtained regarding the effect of straining rates on the diagonal tensile strength of concrete.

5.3 Reinforced concrete Elements [54,68-70,79-82]

Most of the research carried out on the effect of rate of strain has been done on flexural members. In this case, the main effect has been an increase in the yield moment as it should be expected according to the behavior of each of the component materials under high straining rates. A series of tests have been carried out recently at Berkeley [79] to study the effects of repeated reversal loadings at different rates on uncracked and repaired reinforced concrete members.
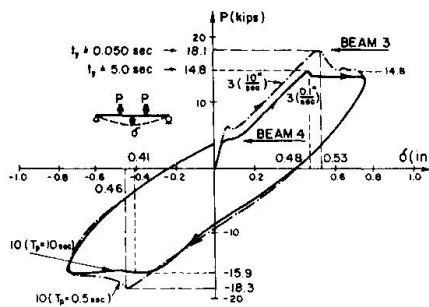
Some results are illustrated in Fig. 13. The results show that: (1) the only significant effect of high rates of strain is the increase of the first yield moment; (2) when the deformation is increased beyond that required for first yielding, the effects of rate of loading become less and less; and (3) after a cycle of loading in which the member is subjected to deformation beyond its first yielding, the hysteresis loops are little affected by the straining rate.

FIG.13 - EFFECT OF LOADING RATE

5.4 Prestressed Concrete Elements [54,83-85]

Most of the tests have been carried out on beams, although there have been some tests on frames [83]. The results of all these tests show that as the rate of loading increases, the moment resistance increases, and the increases are largest at the elastic limit and smallest at rupture. In other words, the results are similar to those found for ordinary reinforced-concrete elements.

5.5 Significance of Results Available and Conclusions

The question that designers would like to see answered concerns the significance of the available test results on the design of structures against real dynamic excitations. It should be noted, in answering this question, that it is the response of the whole structural system, rather than the frequency content of the excitation, which determines the dynamic effect felt by its critical regions and their component materials. For example, in case of earthquake, the ground motion might have very high frequencies, but the rate of straining at the critical regions of the structure will depend on the dynamic characteristics of the structural system, especially on its fundamental period of vibration. Therefore in evaluating for seismic design the possible effect of rate of straining it is necessary to consider the range in which the fundamental period of actual structures can vary. In the case of very rigid structures with very short periods, 0.1 sec. for example, the rate of straining at the critical regions can be as high as 2.5 in/in/sec. or even more. On the other hand, in case of very tall and slender structures in which the fundamental period can be as long as 10 sec., it is clear that the rate of straining will be very low, perhaps only 0.025 in/in/sec.

Therefore, it is clear that if we can expect some significant effects from rate of straining during earthquakes, they might occur only in rigid structures. Due to the fact that, even in this case, the rate of straining will be only about 2.5 in/in/sec., the only significant effect this rate would have is an increase in the first yield-moment resistance at the critical regions, which might amount to about 25%. Since this is the only apparent beneficial effect, most investigators and engineers agree that it may be conservative to use just the pseudo-static test results as a basis for design. While this will be

certainly true in cases of members subjected to practically pure flexure, it might not be true in cases where shear and axial forces are important. A 25% increase in moment capacity might result in a 25% increase in the shear forces acting at the critical regions. Then the question is, whether the rate of strain that has induced a 25% increase in moment capacity also produces an increase in the shear resistance of these critical regions. Because of a lack of reliable information about the effect of rate of strain on the shear resistance of critical regions, the author feels that it would be desirable to design the web reinforcement of these critical regions with consideration for the possible increase in moment capacity of these regions, according to the maximum rate of strain probable.

There are already available results from series of tests on models of structural elements and simple frame structures which in general confirm that the only important effect of the rate of strain that could be induced in extreme earthquakes is a possible increase in moment capacity [80-82]. In evaluating the results of tests on models, it should be kept in mind that the smaller the model, the higher the frequency range for the model test. Thus in small models, the rate of strain could have a significant influence on such material characteristics as the strength, the modulus of elasticity, internal damping etc. This influence should be considered in interpretation of the test results.

6. PROPOSED MODELS FOR LOAD-DEFORMATION RELATIONSHIPS

Guided by some of the experimental results that have been reviewed here, several investigators have suggested models for the load-deformation relationships of reinforced concrete incorporating the observed changes in stiffness with cyclic loadings [25,31,32,56,65,80,82,86]. Although these models have been proposed for dynamic analysis of seismic response, they are based on static force-deformation relationships. While some of the proposed models [25,31,32,65] have been derived theoretically from the static stress-strain relationships of the materials under cyclic straining, taking into account the main factors that contribute to the stiffness degradation, other models can be constructed only by following a series of rules [82,86]. These rules have been established from direct analysis of results obtained in static tests. One of the most significant contributions in this area is the work of Takeda et al [82]. These investigators have checked their static force-displacement model against a series of static tests and have then used their model to predict responses with those measured in a series of tests carried out on a shaking table. The agreement was excellent, except that measured maximum accelerations exceeded the calculated values. The main reason for this discrepancy was that, as the tests were carried out on models, the strain rate was very high and therefore affected their resistance. A third group of investigators have suggested that the experimental hysteresis loops can be expressed as polynomials in terms of two or three test parameters [56,80].

All the proposed models are for frame structural elements and are based on the assumption that the failure is controlled by ductile flexural behavior. No model has been suggested for frame elements in which the failure is controlled by shear, or for the case of shear wall elements and subassemblages. These are areas in which further research is needed.

7. CONCLUDING REMARKS

Modern testing techniques and facilities have permitted investigators to appreciably improve the understanding of the behavior of concrete structures under generalized excitations, of which repeated forces is one particular case. However, from the review and discussion presented here, it is clear that a lot remains to be learned regarding the actual behavior of concrete structures under this type of excitation. Information, and therefore experimental work is needed

in the following areas:*

1. Thorough instrumentation of a large number of real structures and their surroundings, to obtain reliable data regarding the history of actual excitations, the dynamic characteristic of the structures (T and ξ), and their response.

2. To further the dynamic and static testing of real structures or of large scale models under realistic sequences of excitation, to study the interaction of the different structural components, as well as how they interact with the non-structural elements. The variation of T and ξ with increasing damage should be studied. The sequence of excitations should be determined through integrated (iterative) experimental and analytical studies.

3. According to range on which the sequence of actual excitations on structures can vary, to establish well defined loading conditions to test conservatively real structures and/or large-scale models and their elements and sub-assemblages.

4. To continue studying the behavior of flexural critical regions to establish better criteria for delaying buckling of the main reinforcement.

5. To develop a comprehensive program for testing flexural critical regions with high shear and with combined high shear and axial forces. Standard ways of testing should be established, to facilitate comparison and interpretation of results. Emphasis should be placed on the evaluation of degradation in strength, stiffness, and energy dissipation with the number of cycles.

6. Experimental studies on realistic beam-column assemblies, including the contribution of floor slabs and walls, should be initiated. The effective widths of these elements should be determined from a point of view of stiffness as well as strength. Emphasis should be given to the mechanism of failure at the joint panel zone.

7. Studies are needed on the behavior of different types of shear walls with various boundary frames, with the main objective of improving their ductility and energy dissipation capacity, and to formulate mathematical models for their hysteretical behavior.

8. The effect of rate of straining, especially in the case of combined states of stress, needs to be determined.

9. Investigations should be made of repair methods on structural components that have been damaged during high intensity excitations. The strength, stiffness and energy dissipation capacity of these repaired components should be compared to that of the original elements.

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 anchorages under dynamic loading, etc., still remain to be answered.

It is hoped that this introductory report can serve as a basis for assessing priorities in preparation of papers and discussions at the symposium, and that the symposium will contribute to the resolution of many of the problems and questions that have been raised here. International collaboration of engineers and researchers is needed, especially in defining some standards for conducting the experimental work that is needed, and in coordinating human and economic resources toward a closer and more realistic approach to the solution of the problems regarding structural safety that face the construction industry.

8. ACKNOWLEDGEMENT

* Reference [3] presents a detailed discussion of research needs for further advancement in the field of design of reinforced concrete structures.

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SUMMARY

The problems involved in the design of a structure are first discussed. The intimate interrelationship between the mechanical behavior of the structure and the actions that are exerted on it is emphasized. The sources, treatment and effects of the different types of actions are summarized. Then the experimental studies that have been carried out regarding the behavior of reinforced and prestressed concrete structures and their elements under generalized actions are reviewed. The significance of the available test results are discussed. The effects of rate of straining are reviewed and finally the areas in which experimental work is needed are indicated. Close interaction between experimental and analytical studies is highly recommended.

RESUME

Les problèmes relatifs au projet et calcul d'une construction sont d'abord présentés. L'accent est mis sur la relation étroite existant entre le comportement mécanique de la construction et les forces qui s'exercent sur celle-ci. Un résumé des causes, solutions apportées et effets de différents cas de charge est présenté. L'ensemble des études expérimentales du comportement des constructions en béton armé et précontraint et de leurs éléments dans des cas de charges généraux est passé en revue. La valeur des résultats de mesure disponibles est analysée. Les effets de la vitesse d'élongation sont discutés et les domaines dans lesquels un travail expérimental s'avère nécessaire sont indiqués. Il est hautement recommandé de garder un contact étroit entre les études expérimentales et analytiques.

ZUSAMMENFASSUNG

Vorerst werden die Probleme, die in der Bemessung von Bauwerken auftreten, besprochen. Die enge Beziehung zwischen dem mechanischen Verhalten der Bauwerke und den Beanspruchungen, denen sie ausgesetzt sind, werden nachdrücklich betont. Die Ursprünge der verschiedenen Arten von Beanspruchungen, deren Behandlung und Wirkung werden zusammengefasst. Sodann wird ein Ueberblick der experimentellen Studien vorgelegt, die über das Verhalten von Stahlbeton- und Spannbetonbauwerken und deren Bestandteilen unter allgemeinen Belastungen durchgeführt wurden. Ferner wird die Bedeutung der vorhandenen Versuchsergebnisse erörtert. Schliesslich werden die Wirkungen der Dehnungsgeschwindigkeit besprochen und die Gebiete, auf denen Versuchsarbeit erforderlich ist, angegeben. Eine enge Beziehung zwischen experimentellen und analytischen Studien wird nachdrücklichst empfohlen.

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