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Dynamic Behaviour of Reinforced and Prestressed Concrete Buildings under Horizontal Forces and the Design of Joints (Incl. Wind, Earthquake, Blast Effects)

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1. Introduction

This report covers two major aspects of behavior and design of reinforced and prestressed concrete buildings under dynamic loading such as wind, earthquake, and blast: (1) A summary of the present status of knowledge, and (2) a discussion of requirements for further advancement in knowledge. The term "building" as used herein encompasses a wide range of structures including conventional frame buildings, shear wall structures, curvilinear structures, reactor containment vessels, as well as other types of structures. Methods of design for dynamic loading have generally been based on empirical approaches or conventionalized static analogs, since rational techniques of analysis for dynamic loading for fixed structures have only recently become widely used. As is the case with statical design of structures, in dynamic design the greatest problems arise with the joints and connections between elements, which for reinforced concrete or prestressed concrete buildings means the details of placement and anchorage of reinforcement.

Before proceeding with a discussion of the strength and ductility requirements for dynamic loading, and with the choice of structural layout and framing, one must consider the design philosophy that is to be followed, the relationship among the choices of method of analysis, design parameters, the hazards for which the design is to be made, the frequency of possible occurrence of loadings or the probabilistic nature of the loadings expected, and the adequacy of approximations employed in relation to the allowable margins to provide for uncertainties or lack of knowledge.

Design Philosophy

A structure may span its entire useful life without once being subjected to a major or even to a moderate earthquake. The provision of resistance against earthquakes in the design of the structure may be considered to be a form of insurance. Reasonable amounts of insurance are necessary to guard against the incalculable costs of personal injury or loss of life, or to be commensurate with costs of physical repair and renovation if damage occurs. To be consistent with the latter requirement implies that the margin of safety in design against earthquakes should be sufficient to minimize the total of the additional design costs and the cost of repairs for earthquakes of normal maximum intensity during the life of the structure. However, under all circumstances the margin should be sufficient to avoid calamitous failure with attendant loss of life or major personal injury even if the extreme maximum intensity of earthquake that can be expected in the region should occur. This philosophy implies different margins of safety for different types of structures.

A similar philosophy governs design to resist wind loadings. The normal maximum expected intensity of wind loading may occur more often during the life of a structure than the normal maximum expected intensity of earthquake, and therefore may require a greater relative margin of safety. However, the provision for the extreme wind condition involves the same type of consideration as that just described for an extreme earthquake.

Other lateral loadings can be related to those of earthquake or wind. Earthquake loadings correspond primarily to ground motions imparted to the base of the structure, and wind loadings to forces transmitted to the aboveground portions of the structure. Blast loadings can involve both of these aspects, including the air overpressure forces which are similar to wind loadings, and the ground motions accompanying either buried or contact blasts, which produce ground motions as well. Impact loadings from aircraft and "sonic boom" loadings are of a somewhat different nature but can be related in general to the effects of the other types of loadings described.

All of these loadings have the characteristic that they are governed to at least some degree by probabilistic considerations, and the duration, the intensity, and even the time variation of the loadings are subject to the laws of chance. These characteristics of the normal types of lateral loading require

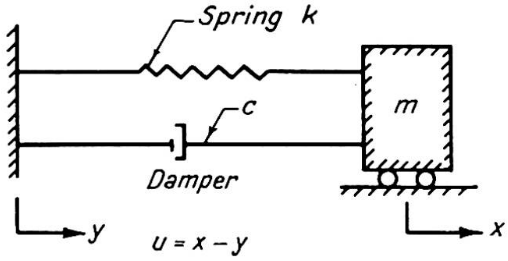

Fig. 1. Simple Linear Oscillator

additional study; it may be completely unrealistic to treat them as deterministic forces and motions.

2. Present Status of Knowledge

The Reponse Spectrum Concept

The simplest means of reviewing the over-all strength and ductility requirements of an earthquake resistant design involves the concept of the response spectrum. The response spectrum for earthquake ground motions is a plot against the frequency of vibration of the maximum response, attained either during or after the input motions, of a simple linear oscillator such as shown in Fig. 1, whose base is subjected to the earthquake motion history. A typical response spectrum, that corresponding to the most intense earthquake for which adequate records exist, namely the El Centro Earthquake of May 18, 1940, in the north-south horizontal component of motion, is shown in Fig. 2.

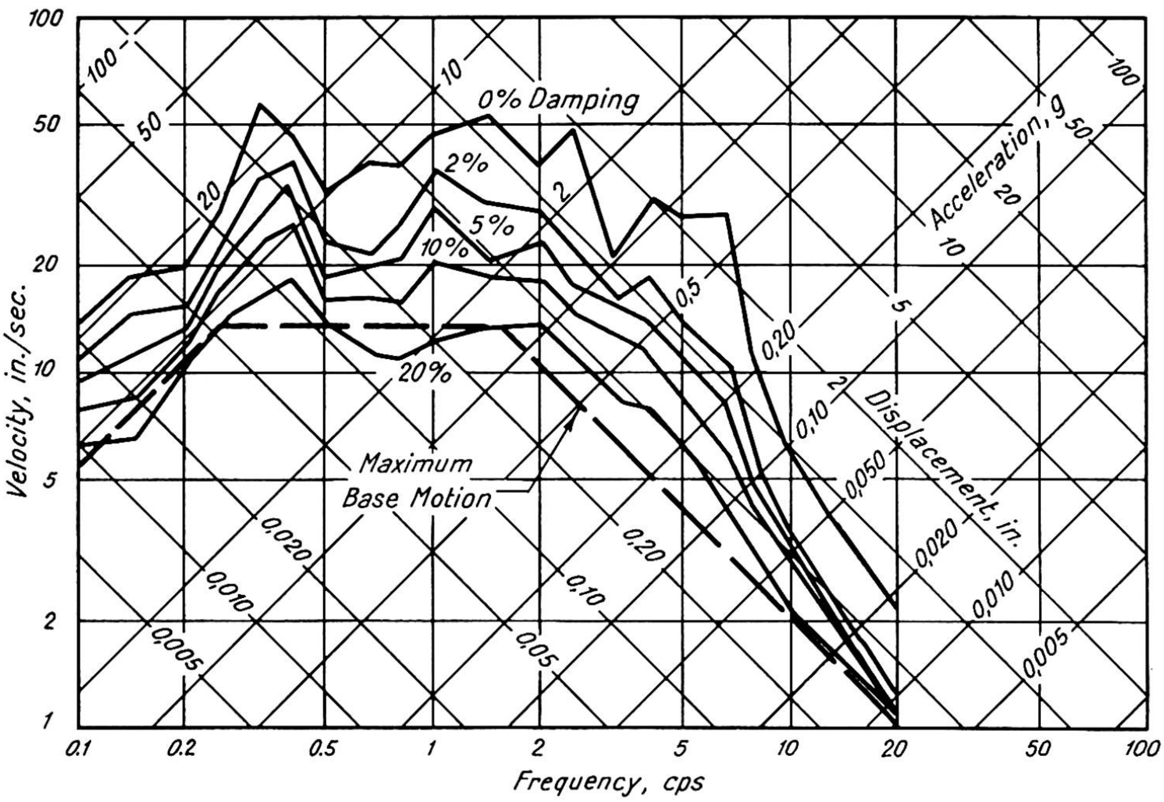

Fig. 2. Response Spectra for El Centro Earthquake of May 18, 1940, N–S Component

The choice of a logarithmic scale for frequency of the linear oscillator simplifies the response spectrum and permits the simultaneous plotting of three related quantities which define a number of aspects of the maximum response of the system; the scales are noted on the figure. The maximum displacement of the mass relative to the ground, D, which is a measure of the maximum strain introduced in the spring by the earthquake motion, is plotted with reference to the lines which slope up from left to right. The maximum acceleration of the mass, which is a measure of the maximum force generated in the spring, is plotted with reference to lines which slope down from left to right. What is actually plotted is not the actual maximum acceleration, but something that is nearly equal to the maximum acceleration, more accurately described as the maximum pseudo acceleration, A, defined by the relation:

$$A = \omega^2 D \tag{1}$$

where ω is the circular frequency of vibration of the linear oscillator.

The maximum energy stored in the spring is given by the quantity

$$\tfrac{1}{2} m V^2$$

where m is the mass of the oscillator, and V is the maximum pseudo relative velocity, which is not always quite the same as the maximum relative velocity, but which has the dimensions of velocity, and which is related to the maximum relative displacement D by the formula:

$$V = \omega D \tag{2}$$

The simultaneous values of the quantities A, V, and D are given by the one curve on the figure for a particular value of relative damping in the system, where the damping, relative to the critical value of damping, is given by the quantity β. Curves are indicated for the earthquake response spectrum in Fig. 2 for values of 0, 2, 5, 10, and 20 percent of critical damping.

Where the spring constant of the linear oscillator is k, and the period of vibration is T, the natural frequency, f, is given by the relation

$$f = \frac{1}{T} = \frac{1}{2\pi} \sqrt{k/m} = \frac{1}{2\pi} \sqrt{g/u_s} \tag{3}$$

where g is the acceleration of gravity, and u_s is the static deflection of the spring produced by the weight of the mass if it were to hang on it vertically.

The response spectrum can be drawn as a function of period as well as of frequency. This results merely in reversing the diagram end for end without changing its shape.

It is noted that for dynamic motions of the base of the simple oscillator, such as that arising from earthquake, blast motions, or simple base motion pulses, the response spectrum has the characteristic that for very low frequencies, the maximum response displacement, D, is virtually constant and is equal in value to the maximum displacement of the ground, d_m. For very high frequencies the maximum pseudo acceleration of the mass, A, is virtually constant and is practically equal to the maximum acceleration of the ground, a_m. For intermediate frequencies, the maximum response displacements, velocities, and accelerations are all amplified over the ground motion maxima, with the factors of amplification being a function of the proportion of critical damping β. For values of β in the range of about 5 to 10 percent, these amplification factors are, respectively, for displacement, velocity, and acceleration, slightly over 1, about 1.5, and 2.0. Methods of using the response spectrum for the analysis of single and multi-degree-of-freedom systems are described in various references [Refs. 1, 2, 3, 4, 5].

Because of the relation between the response of dynamical systems to motion or to external loading, there is an equivalence between the intensity of external loading, and the inertial loading, $-ma$. Hence, one can draw a diagram similar to the earthquake response spectrum for loadings such as wind. It is convenient to use, for this diagram, force or pressure instead of acceleration, and impulse instead of velocity. With this designation, the asymptote for the high frequency end of the diagram for wind loading approaches a value corresponding to the maximum wind load intensity. There is no corresponding bound for the low frequency end of the spectrum; for intermediate frequencies, the bound probably does approach a horizontal asymptote the value of which is determined by the average pressure multiplied by the total wind duration. A possible sketch of a wind response spectrum is shown schematically in Fig. 3.

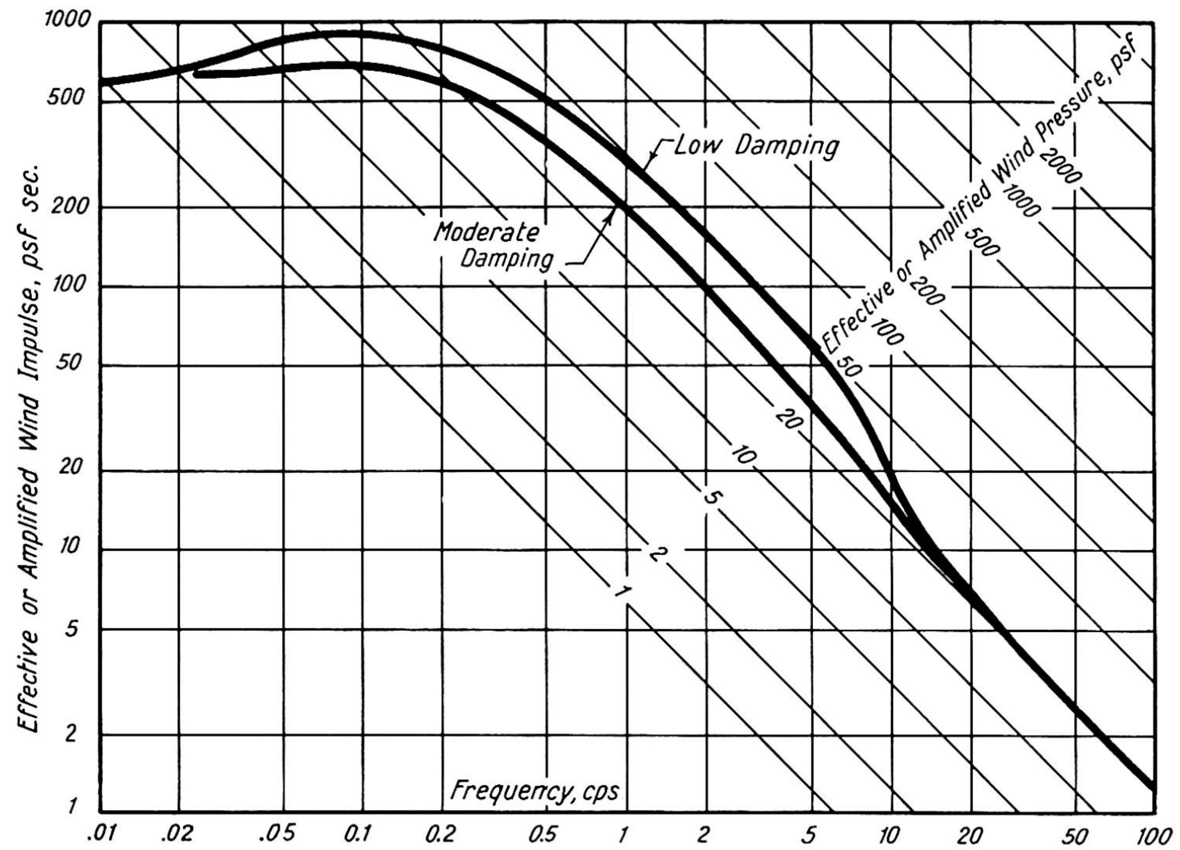

Fig. 3. Possible Sketch of Wind Response Spectrum

Strength and Ductility Requirements

From Figs. 2 and 3 it is apparent that the intensity of force and the amount of energy that must be absorbed in a dynamical system depends on the intensity of earthquake motion or the intensity of wind force, but is affected materially by the energy absorption within the structure itself, corresponding to damping, and arising from interaction of the various parts of the structure, especially the partitions and other non-structural parts reacting with the structural components, energy losses due to the coupling of the structure with its foundation, and also energy losses due to inelastic behavior of the structure in its response.

It is of special interest to consider how inelastic behavior affects the response spectra described previously for elastic behavior. If one considers that the spring of the linear oscillator has an elasto-plastic characteristic with a yield point, where the maximum permissible deflection is related to the deflection at the limit of elastic behavior by the ductility factor, μ, as shown in Fig. 4, then in general for inelastic behavior, as described in Refs. 6, 7 and 8, the response has the following characteristics:

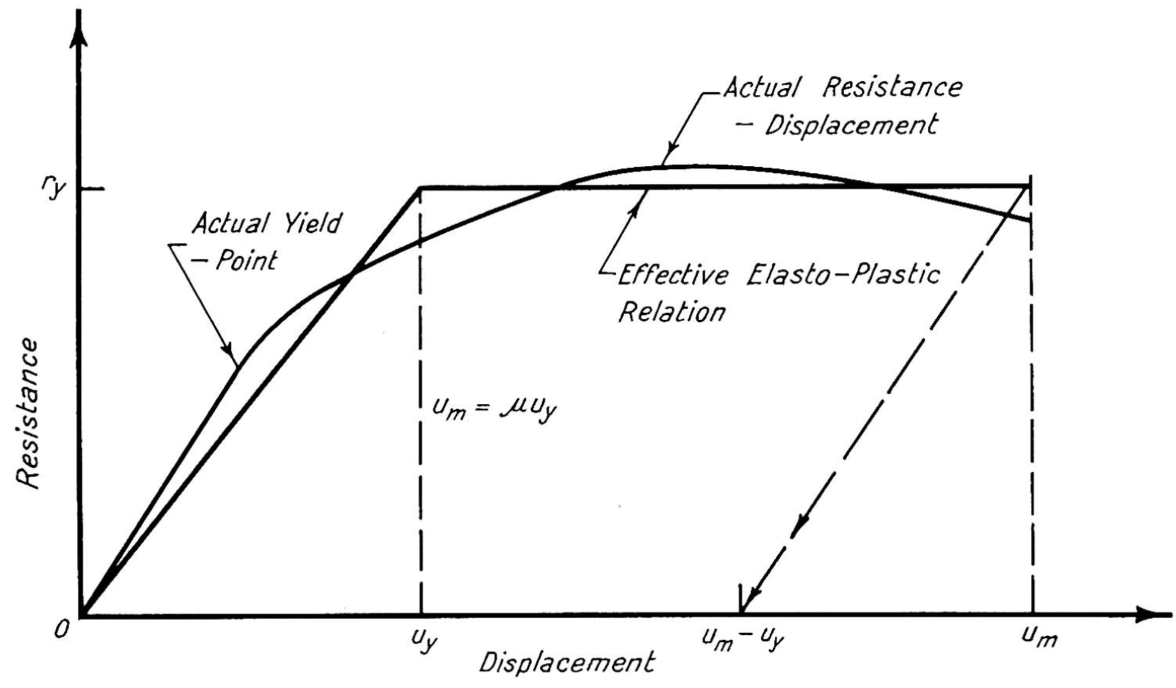

Fig. 4. Elasto-Plastic Resistance-Displacement Relationship

(1) For the high frequency part of the spectrum where the acceleration response is virtually constant, the force is about the same for both the inelastic system as for an elastic system having the same initial frequency.

(2) In the middle part of the spectrum, for intermediate frequencies where the velocity response is virtually constant, the total energy absorbed in the inelastic system is about the same for an elastic system having the same frequency.

(3) For low frequency systems, where the displacement response is nearly constant, the displacement of the system is about the same for the inelastic system as for an elastic system. In some cases there are bounds which limit the response to even lower values than those corresponding to the foregoing rules. Even where force is nearly preserved, the energy can never be increased over the maximum value for the corresponding elastic system, and even where absorbed energy is nearly constant, the displacement can never be greater than the maximum relative displacement for the elastic system.

For tall buildings, as indicated by the appropriate range of frequencies of about 0.2 to 4 cycles per second, from Figs. 2 and 3 it is seen that the range of behavior is generally in that range where either energy or displacement is preserved for earthquake responses, or where the force is preserved for the wind response problem. Hence, inelastic behavior under wind loading does not generally reduce the response. However, inelastic behavior under earthquake loading may reduce the forces materially for which the structure must be designed.

One obviously has several choices in the design of a structure to resist earthquakes. He may choose to make the structural resistance high and keep the structure virtually elastic. On the other hand, he may choose to make the energy absorption capability of the structure very high, and permit the structure to deform inelastically. If he chooses the latter alternative then he can design the structure for a considerably lower force than would be required for the former case; of course, he accepts the penalty of possibly having a deformed building. However, the amount of deformation may be acceptable if it does not correspond to a collapse condition.

In other words, one must balance the strength with the ductility required in the design of a building to resist earthquake motions, but one must have the necessary strength in all cases to resist wind forces. However, it may be necessary, because of the difficulty in achieving very large amounts of ductility, to increase the lateral force level for which the design is made to account for the lateral forces mobilized by the earthquake motions. In both instances, it is essential that the construction process be adequately controlled and inspected to insure that the required strength, and above all the required ductility, can be mobilized without premature failure.

Because of the penalty involved in providing for the extremely high strength requirements to resist major earthquakes, it is customary to design for considerably lower forces than the theoretical values that would be found in a purely elastic system. Hence most building codes imply a ductility factor of the order of 4 to 6, and in some cases possibly even higher, which must be maintained by the proper design details as well as adequate inspection and control of construction. This topic is discussed in detail in Chapter 6 of Ref. 1.

Structural Layout and Framing

Buildings can be designed with different combinations of components. Common buildings of reinforced concrete can be made up of beam and column construction constituting a flexural framework. They may have solid walls or shear walls offering lateral resistance as a vertical cantilever beam; or they may be made up of combinations of frame and shear walls acting together. However the layout is made, resistance must be offered to torsion of the buildings caused by nonuniform distribution of resisting forces and masses, by accidental irregularities, or most commonly by variations in the intensity of motion over the plan of the building foundation.

The different types of construction have different inherent strengths and ductility factors. In general, beams without axial compression have the highest ductility, where, under conditions of proper design and construction these factors may approach values of 10 or more; columns or flexural members with high values of compression have somewhat lower ductilities. If the compression forces approach the compressive strength of the member, the ductilities can be very low. However, with adequate arrangements of reinforcement, the ductility in columns can be made as high as 5 to 6 or more. However, in tension the situation is quite different and the strength is in many cases seriously lower relatively than under compressive conditions in reinforced concrete. Hence, attention must be paid to the elimination of major tensile forces over the gross section of a member.

In order to attain higher amounts of ductility, shear failures and compressive failures in concrete flexural members must be avoided. This means that compressive reinforcement must be used or a limit must be put on the difference between the amounts of tensile reinforcement and the compressive reinforcement at a cross section; and shear or web reinforcement must be used to provide resistance against diagonal tension cracks. Adequate anchorage of reinforcement to avoid bond or anchorage failures is also required. These topics are discussed in detail in Chapter 5 of Ref. 1.

Ductility of Beams

The load-deformation characteristics of reinforced concrete members have been studied in several investigations at the University of Illinois, beginning in 1951. Studies of load-deflection properties of simple-span beams loaded at the third-points were reported in 1952. Additional tests of simple beams loaded at mid-span through a stub, to simulate a beam-column connection, were reported in 1954.

More recent tests were begun in 1960 and completed in 1962. The object of this study was to determine the amount of rotation and ductility that can be developed at the connection between monolithically cast reinforced concrete beams and columns, and to develop procedures for predicting the moment-rotation characteristics of such connections. A summary of the results of the study, as adapted from Ref. 9, follows. The effects of the following variables were studied:

a) The depth of the member.
b) The presence of compressive reinforcement in various amounts.
c) The effects of unloading and reloading the member several times, at various levels of load or deformation.
d) The effect of reversing the load or moment, again at various levels of load or deformation.
e) The effect of axial load on the member, such as would be present in a column.

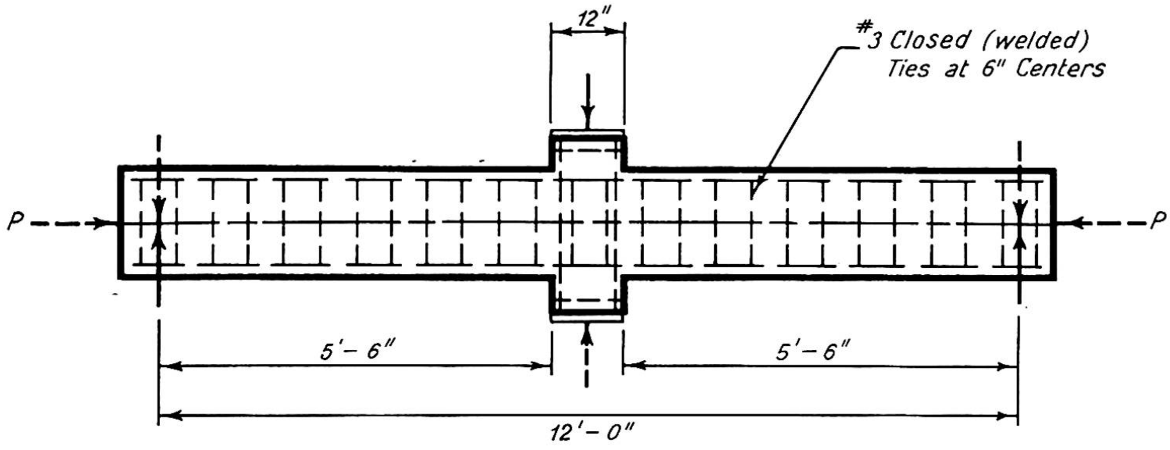

Fig. 5. Elevation of Test Specimens

The type of specimen used is shown in Fig. 5. The members were supported at the ends on a 12-ft. span, and were loaded transversely through the stubs at mid-span. Seventeen specimens were tested as beams (that is, with no axial load) and 11 specimens were tested as columns with an axial load, indicated as P on the figure, applied at mid-depth and held constant while the transverse load was applied. The tests with and without axial load will be discussed separately. All specimens were provided with transverse reinforcement in the form of No. 3 deformed bars welded into a closed loop and spaced, usually, at six inches.

Deflections were measured at mid-span and at other locations along the span, and numerous strain measurements were made on the reinforcement and on the concrete, using both electrical resistance strain gages and mechanical gages.

The strengths of the concrete and the reinforcement were not major variables in the tests. The cylinder strength of the concrete at the age of tests was usually in the range from 4 to 5,000 psi. The reinforcement was intermediate grade deformed bars with yield strengths between 45 and 50,000 psi.

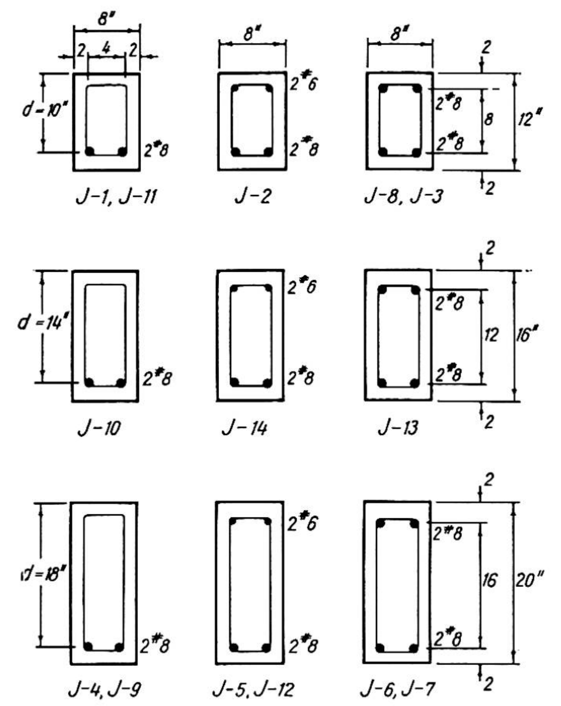

Fig. 6. Cross Sections of Test Beams

The cross-sections of the beam specimens, without axial load, are shown in Fig. 6. The principal variables were the effective depth and the amount of compression reinforcement. The effective depth of 10 in., used for the five beams at the top of the figure, corresponds to that used in the previous investigations at the University of Illinois and in tests made by Ernst at the University of Nebraska. The other beams had effective depths of 14 and 18 in.

All of the beams had exactly the same amount of tension reinforcement. However, the amount of compressive reinforcement ranged from zero to an amount equal to the tension reinforcement, with an intermediate value about half as large.

The curves in Fig. 7 give a fairly good picture of the type of behavior observed in these tests. Although they are plotted in terms of load and deflection, there is a direct relation between load and moment, and also between deflection and rotation at the stub, for the type of specimen tested. The variable for these curves is the effective depth of the beam; all other properties were substantially the same.

The first sharp break in each curve corresponds to yielding of the reinforcement. As would be expected, the load at which yielding occurred varies directly with the depth. The increase in load beyond yielding results primarily from strain-hardening of the reinforcement. So far as these curves are concerned, the next significant stage in the response of the member is the break at a deflection of 10 to 12 inches. This break represents the ultimate moment and deflection. Although there is an increase in ultimate moment with depth, as would be expected for a constant amount of tension reinforcement, there is relatively little difference in the ultimate deflections.

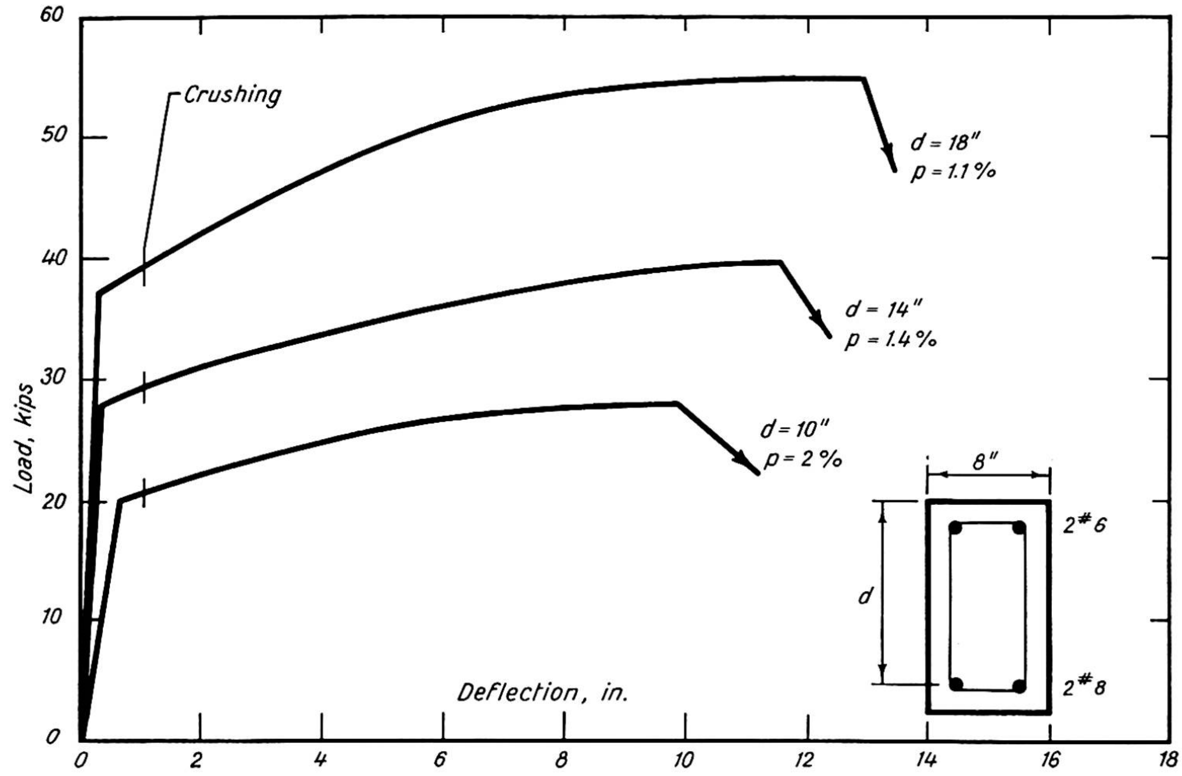

Fig. 7. Effect of Depth on Load-Deflection Characteristics

The cross marks on the curves at a deflection of about one inch represent the stage which is loosely called "first crushing," and which corresponds to the first visible signs of spalling or crushing of the concrete on the compression face. In terms of the structural response of the member, this point has little significance, as is evident from the fact that the curves are continuous through it. However, from a theoretical or analytical point of view, this stage corresponds to the development of compressive strains in the concrete of about three-tenths or four-tenths of a percent, and the moment at this stage thus corresponds to what we would compute as the "ultimate" moment by the conventional ultimate strength theories like those presented in the ACI Building Code.

At "first crushing" the damage is extremely small and, although it may affect slightly the appearance of the member locally, it has no significance in relation to the structural response of the connection. From this point of view, it is the ultimate deflection and moment which are important.

Figure 8 shows the effect of adding compression reinforcement. Although the relatively large influence of compression reinforcement on the ductility of beams has been known for some time, it can never be emphasized too much. Where ductility is desired, the addition of suitable amounts of compression reinforcement is still the most effective way to provide it.

Beam *J–10* in Fig. 8 was reinforced in tension only. Since the steel percentage was relatively small, this beam had a fairly large amount of ductility; the deflection at ultimate was about eleven times as great as the deflection at

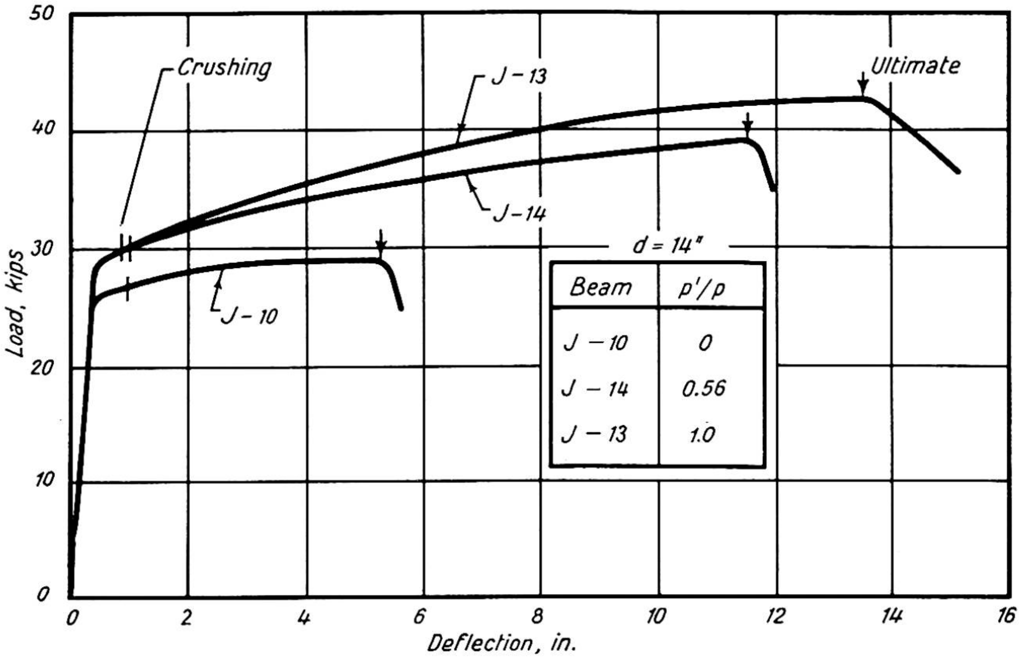

Fig. 8. Effect of Compression Reinforcement. $d = 14''$

yield. Even so, the ductility in this series of beams was increased considerably by the addition of compression reinforcement. Beam *J–14* had approximately half as much compression steel as tension steel, and in Beam *J–13*, the top and bottom steel were the same.

Although the loads and deflections at yield and at first crushing were practically the same for the three beams, the addition of compression reinforcement increased significantly both the ultimate load and the ultimate deflection. The increase in ultimate load resulted primarily from strain-hardening of the tension reinforcement; steel strains on the order of 10 percent were not unusual. It may be noted that practically doubling the amount of compression reinforcement in Beam *J–13*, as compared to *J–14*, did not produce a comparable increase in ultimate deflection, although the ductilities of both of these beams are obviously more than adequate. The only explanation that can be offered for this is that Beam *J–13* carried the highest load of any beams in this series and the shear corresponding to this load was large enough to cause yielding of the transverse reinforcement. As a result, the failure of this beam involved a faulting or shearing deformation, which one would be tempted to call a shear failure, if it had not occurred at a deflection of 14 in. and at a load about 50 percent greater than the yield load.

Behavior of Beams under Repeated or Reversed Loading

The behavior of these beams under repeated loading will now be discussed. In all of the tests, the load was removed completely and then reapplied at several stages during the test. The load-deflection curve for such a test on a beam without compression reinforcement is shown in Fig. 9. The results indicate that the removal and reapplication of load had little or no effect on either the load carrying capacity or the ultimate ductility.

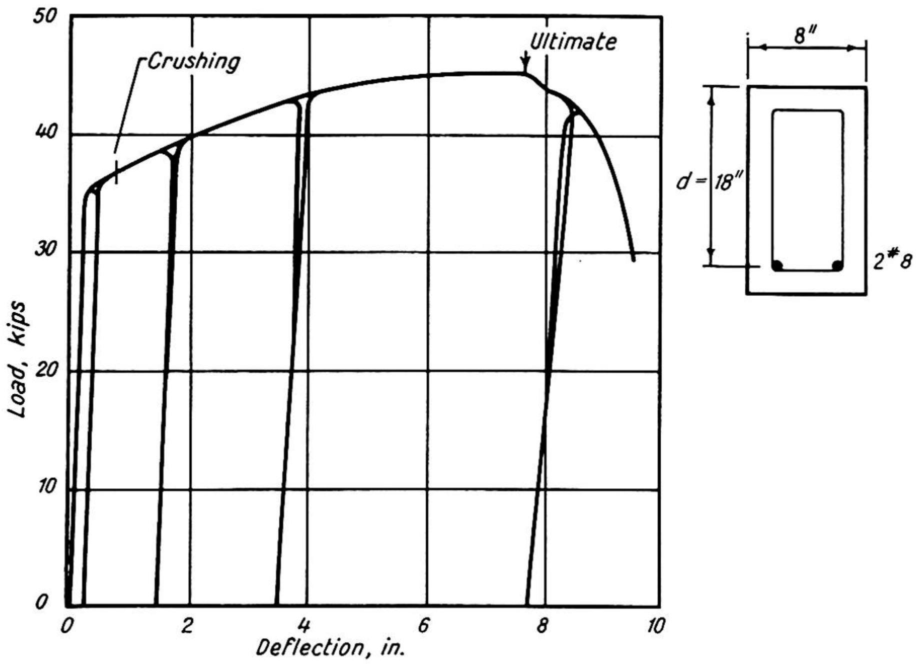

Fig. 9. Load-Deflection Curve for Beam *J–4*

The load-deflection curve for a beam with equal amounts of tension and compression reinforcement showed that unloading and reloading this beam as many as six times in the post-yield range had little or no effect on its load-carrying capacity or ductility.

As can be seen in Fig. 9, after each unloading, the reloading curve was approximately linear up to a load very close to the maximum previously reached. However, the slope of the reloading curve became less steep as the load was reapplied at successively greater deflections. That is, the stiffness of the beam decreased as the amount of plastic deformation beyond yield increased. This phenomenon was amazingly consistent for all of the beams tested.

Three beams were tested under reversals of load, and the load-deflection curve, for one of them, Beam *J–7*, is shown in Fig. 10. This beam had an effective depth of 18 in. and was reinforced equally top and bottom. Also shown on this figure is the *envelope* load-deflection curve for Beam *J–6*; that is, a curve for which the successive unloading and reloading curves have been omitted. This beam was almost identical with *J–7*, but was loaded only in the downward direction. The curve for *J–6*, however, is plotted for both directions of loading for comparison with that for *J–7*.

Beam *J–7* was loaded first downward then upward to loads of about 12, 18, 30 and 35 kips. The latter load of 35 kips represented yield, in both direc-

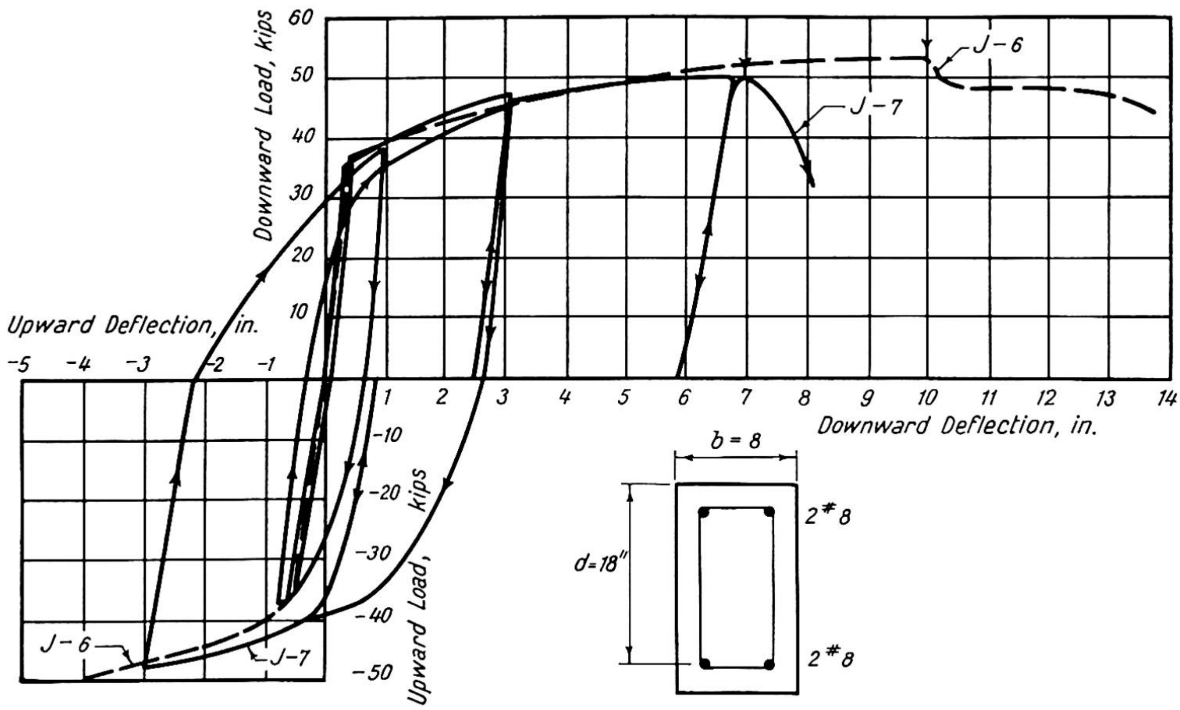

Fig. 10. Load Deflection Curve for Beams *J–6* and *J–7*

tions, since the beam was reinforced symmetrically. It was then loaded to a deflection of about ¾ to 1 inch, in each direction, corresponding to first visible crushing of the concrete on the compression face adjacent to the column stub. The next cycle of reversed loading produced about 3 in. of deflection, first down, then up. And finally, the beam was loaded to failure in the downward direction.

The envelope load-deflection curve for Beam *J–7* compares quite well at all stages with that for Beam *J–6* which was loaded in only one direction. The maximum loads carried were very nearly the same, but the corresponding deflection was somewhat less for *J–7* than for *J–6;* 6.7 in. as compared to 9 in.

The slopes of the reloading curves after a reversal show a definite Bauschinger effect. That is, the reloading curve after reversal of loading is much less steep than the initial load-deflection curve. However, if the beam was unloaded and reloaded in only one direction, without reversal, the reduced stiffness compares very well with those plotted in the preceding figure.

Ductility of Columns

The discussion so far has all been concerned with beams; that is, members without axial load. For this case, it is fairly easy to provide adequate ductility. However, the presence of axial load, as in a column, tends to reduce the available ductility, as shown by Fig. 11. The curve on the left is a conventional interaction diagram of axial load and moment. The solid line represents the combination of moment and axial load which will produce first spalling or crushing

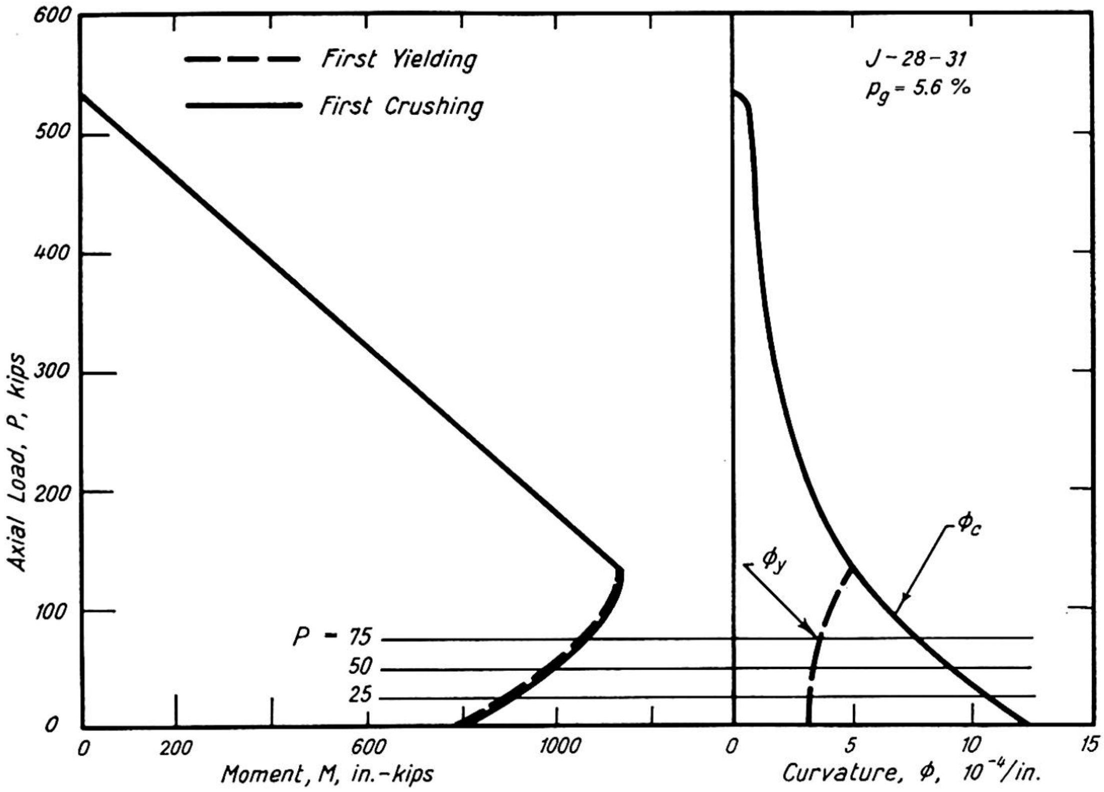

Fig. 11. Interaction Curves

of the concrete as computed by the conventional ultimate strength analysis, and assuming that crushing will occur at a concrete strain equal to four-tenths of one percent. The break in the curve represents the balanced load, above which the concrete reaches crushing strain before the tension reinforcement yields, and below which yielding of the tension steel precedes crushing of the concrete. Below the balance point, the broken line indicates the moment at which yielding of the tension steel occurs. As can be seen, the yield moment and the crushing moment are very close to each other for these assumptions, because the steel is still in the flat yield range when the assumed crushing strain in the concrete is reached.

The curves on the right of Fig. 11 show computed curvatures for the corresponding levels of axial load on the interaction diagram at the left. The solid curve refers to the curvature at the stage which has been called "first crushing," and the dashed curve represents the curvature at first yielding of the reinforcement. Above the balance point, there is only one curve, since yielding of the steel does not precede first crushing of the concrete.

The ratio of the curvature at first crushing to the curvature at yielding can be considered a measure of the ductility; at least, of the ductility corresponding to the stage represented by crushing, although it has been shown that this amount of ductility is only a fraction of that which can be developed before the member fails. Nevertheless, the plots of load versus curvature suggest that the ductility will decrease as the axial load is increased.

Tests were made on eleven members to investigate the effects of axial load on the available ductility. The axial loads used were zero, 25, 50, and 75 kips, which correspond to the levels shown on Fig. 11.

The specimens were of the same general type as those shown in Fig. 5 and their cross-section properties are shown in Fig. 12. In all of the members, the

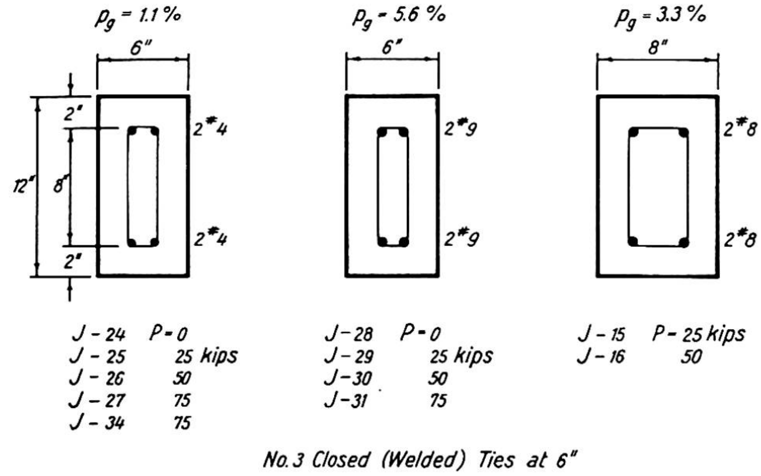

Fig. 12. Cross Section of Beam-Columns (with Axial Load)

reinforcement was equal in the top and bottom faces. There were two main series of tests (the two on the left in the figure) in which the percentage of longitudinal reinforcement was varied from a fairly low value of 1.1 percent total steel based on the gross area of the column, to 5.6 percent. The third series of tests was much more limited in scope, and was simply an extension of one series of the tests on beams, which have previously been described. As

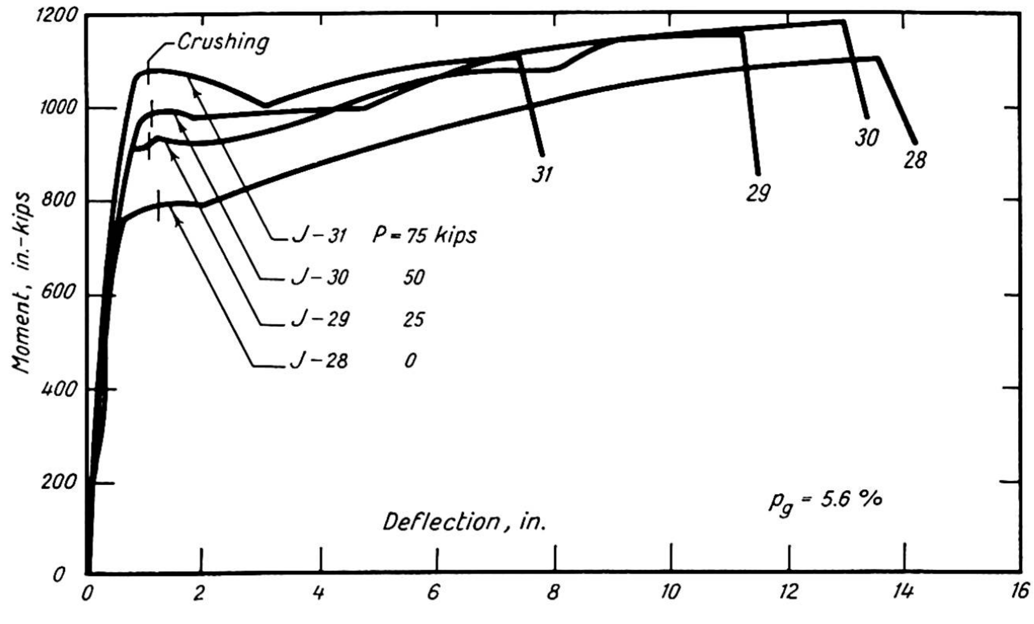

Fig. 13. Effect of Axial Load P on Moment-Deflection Relationship

mentioned earlier, the concrete strength and the yield strength of the reinforcement were substantially the same for all specimens.

The results of the first series of tests are shown in Fig. 13, in the form of moment-deflection curves. The moment shown is the total moment acting at the critical section at the face of the stub; that is, the moment due to the transverse load on the stub, plus the moment due to the axial load acting at an eccentricity equal to the measured deflection.

The yield moment and yield deflection both increase with increasing axial load, as was predicted from the interaction diagram. The crushing moment is very close to the yield moment, and there is a slight tendency for the crushing deflection to decrease with increasing axial load, but this effect is somewhat smaller than was indicated on the interaction diagram. However, as was the case for the members without axial load, first crushing does not constitute a point of any significance in terms of the behavior as represented by these moment-deflection curves. As before, the deflections at ultimate load were many times greater than the deflections at first crushing. However, the increase in load beyond yielding or crushing was usually less as the amount of axial load increases. This is consistent with the effect of axial load in decreasing tensile strains, and thus decreasing the extent to which the steel was strained into the strain-hardening region. And finally, a definite decrease in the ultimate deflection as the axial load increases can be seen, although this decrease is by no means consistent. For example, Beam *J–30* with an axial load of 50 kips had an ultimate deflection greater than Beam *J–29* with an axial load only half as great.

Similar curves were obtained for columns with only 3.3 percent steel on the gross section, and for tests up to a maximum axial load of only 50 kips. The

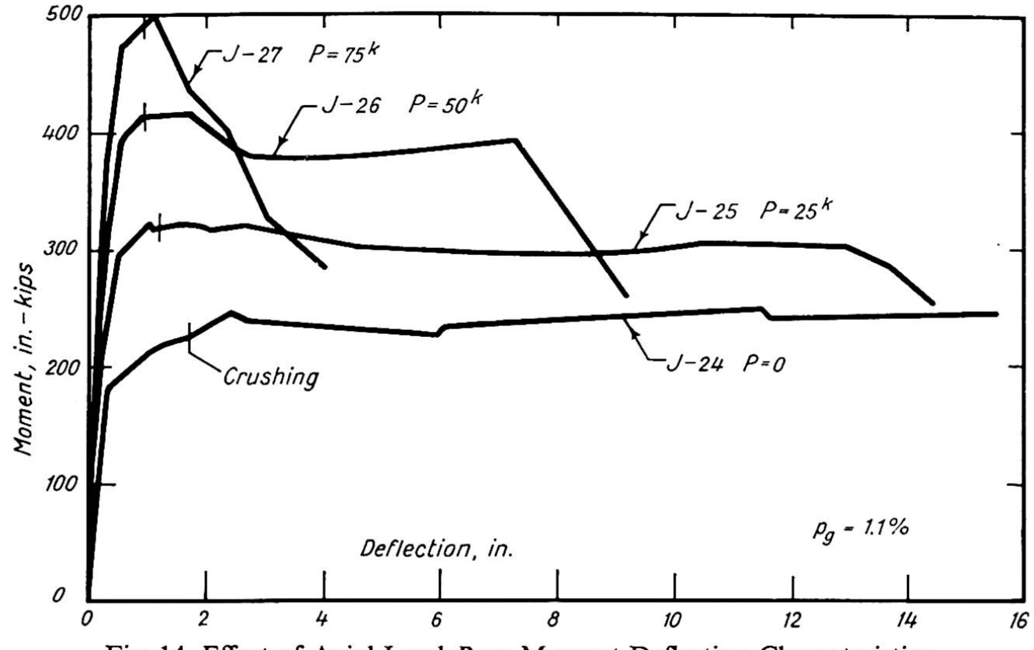

Fig. 14. Effect of Axial Load P on Moment-Deflection Characteristics

results are quite similar to those in Fig. 13. Again, the ductility beyond crushing is very great but tends to decrease somewhat as the axial load increases.

Figure 14 shows the results for a column with only 1.1 percent steel. Although this is hardly a typical case for a column designed for bending under lateral forces, it has been included because it represents a slightly different type of behavior in the presence of axial load. First, it can be noted that the moment tends to decrease beyond the point representing crushing, for the columns with axial loads of 25 and 50 kips. For the axial load of 75 kips, the decrease is quite marked. The reason for this is that the concrete outside the closed ties has spalled off on the top of the beam and on the sides of the beams down to the neutral axis, and the lever arm for the internal resistance has thus been decreased. For the columns with axial loads of 25 and 50 kips, the decrease in lever arm has been offset by an increase in steel stress as it enters the strain-hardening range. However, for Beam *J–27* with an axial load of 75 kips, the steel never reached strain-hardening, and thus the steel stress stayed at the yield point level. Consequently, as the concrete shell spalled off, the reduction in the lever arm was accompanied by a reduction in moment capacity. Nevertheless, this column did not actually fail until a deflection of about 4 in. was reached.

Significance of Ductility Requirements

Reference 1 based its recommendations on the use of a ductility factor, chosen as the ratio of the maximum deflection to the effective yield point deflection, after the representation of the load-deflection curve by an equivalent elasto-plastic approximation. It was concluded that ductility factors of the order of 4 to 6 were sufficient to mobilize the necessary energy absorbing capacity to make effective use of the "Uniform Building Code" procedures. It was felt that the procedures described were conservative since the predicted ductility for the recommended design details was in general considered to be much less than the actual ductility that would be obtained with properly made reinforced concrete structures.

The data presented indicate that this conclusion is indeed correct. For example, Fig. 7 shows ductility factors in excess of 20 to 30, as actually measured. In general, the available ductility in beams or flexural members is more than sufficient. More serious questions arise in members which must carry compressive forces as well as flexure.

It should be pointed out that these difficulties are not limited to any one material. All materials suffer from difficulties in ductility when compressive forces are combined with flexure. Buckling becomes a problem in metals, and crushing is a problem in masonry and reinforced concrete. However, if appropriate attention to details is given, adequate ductility can be obtained in reinforced concrete columns. For example, Fig. 15 shows interaction curves

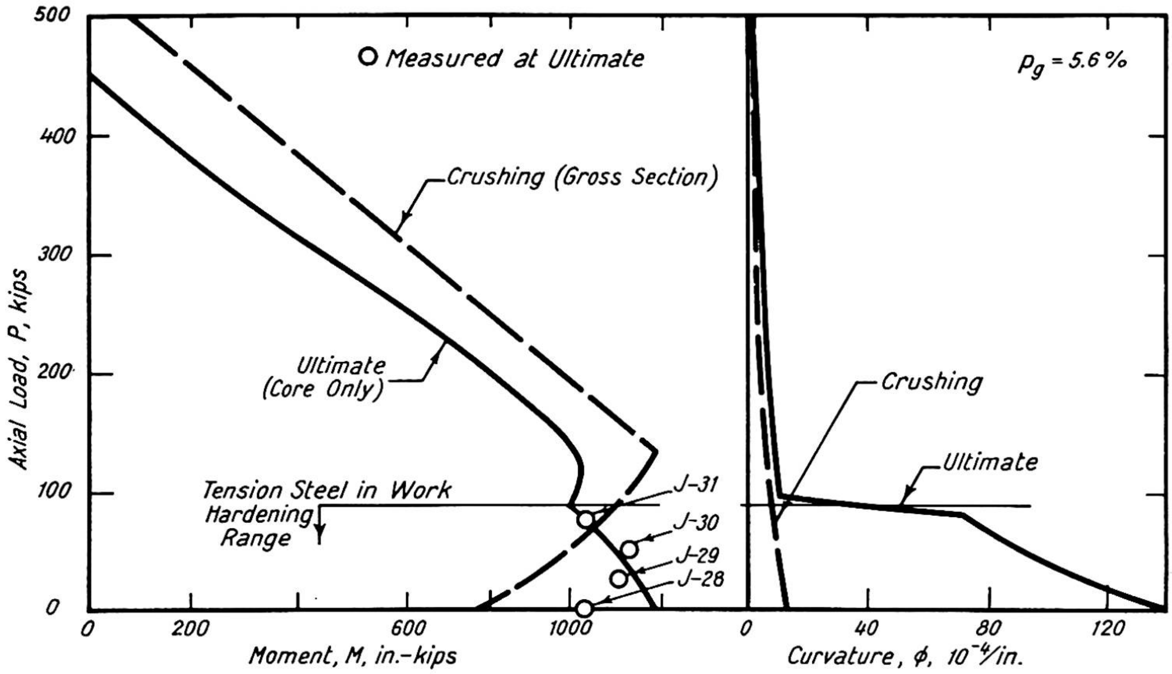

Fig. 15. Interaction Curves at Ultimate

for reinforced concrete columns at ultimate. The curvature at "crushing" is slightly greater than the curvature at initial yielding. The ratio of the curvature at ultimate to that at crushing, which is less than the ductility factor, is seen to be adequately large for values of axial load below the break in the curve, at the "balance" load. Reasonable values of the working stress in compression would correspond to loads not greater than about 50 kips for the column described, which is well below the limit at which the ductility decreases to minimum values.

A further indication of the same conclusion is shown by Fig. 16, where the computed moment-deflection curve and the measured moment-deflection curve are compared. This comparision is shown for a load of 50 kips. Similar results are obtained for other loads, of 25 and 75 kips, for which test data were obtained. It is concluded that the measured deflections and ductilities are considerably greater than the computed values and hence greater than the limiting values required by the design procedures specified in the book.

In Fig. 17, all of the available tests are summarized to give a measure of the ductility factor actually obtained by tests for beams as well as for columns, and a comparison is made with the empirical equation that has been previously used as a measure of ductility of reinforced concrete, namely,

$$\frac{10}{p - p'}, \text{ with an upper limit of 20.}$$

In this equation p is the amount of tensile steel, measured in percent, and p' the amount of compressive steel measured the same way. Hence for two per-

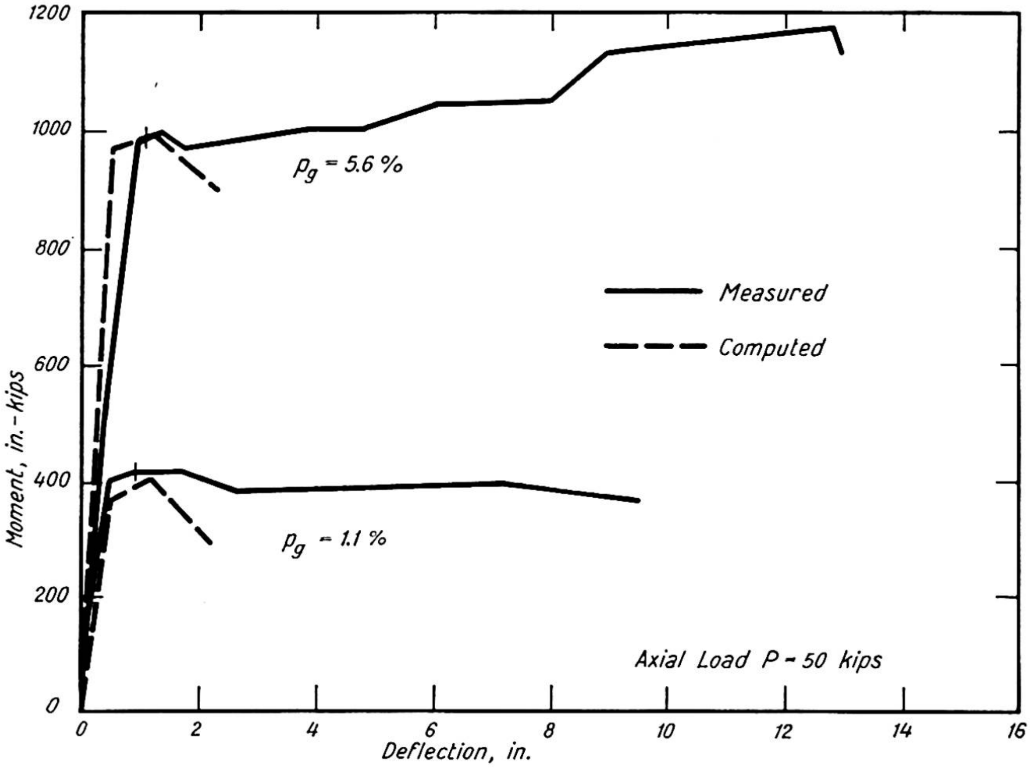

Fig. 16. Measured vs Computed Moment-Deflection Curves

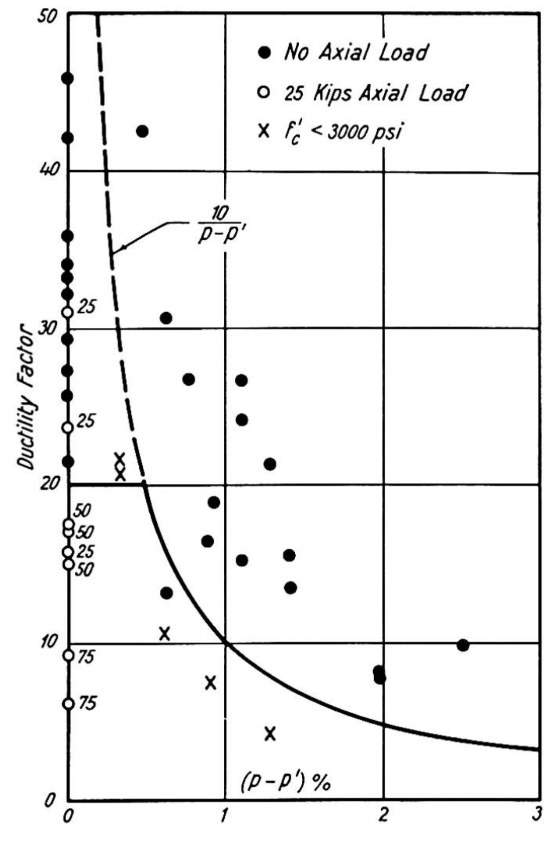

Fig. 17. Ductility Factor for Reinforced Concrete Beams

cent of tensile steel combined with one percent of compressive steel the ductility given by this equation would be 10.

The filled circles on the curve correspond to flexural members with no axial load. In all except one case these are well above the empirical equation. In general, the average values are about twice those given by the equation. The equation is not applicable for zero values of the denominator. It was previously recommended that values of ductility factors no greater than 20 be used for reinforced concrete. This appears to be a reasonably conservative cut-off point for flexural members. Also spotted on the curve are points for columns with axial load, of the proportion shown in Fig. 12. The numerals near the open circles indicate the magnitude of the axial load. For axial loads of 25 kips a ductility factor of 20 is reasonably conservative, although it is not conservative for the higher values of axial loads. Nevertheless, for all axial loads of 50 kips and less the ductility factors were greater than 15, and even for axial loads of 75 kips the ductility factors were greater than 6.

In general, ductility can be obtained in reinforced concrete: (1) if shear reinforcement is provided so that the weakness of the concrete in shear, or rather diagonal tension, is taken into account; (2) if bond and anchorage provisions are properly considered so as to make sure that the reinforcement can act in the way that it is assumed to act; (3) if one avoids too high a proportion of tensile reinforcement, or if one adds an appropriate amount of compressive reinforcement; and (4) if one confines concrete in zones of high compression by hoops, closely spaced ties, or spirals.

Methods of Analysis in Relation to Design

Methods of analysis for use in design for dynamic loadings can be of three greatly different types.

The most involved and complicated type of analysis is that in which one uses typical histories of input motion or loading, varying with time, and computes the response of the structure or the preliminary concept of the structure for which the design is to be made. If one uses a range of inputs to take account of the probabilistic nature of the input motions, or if one makes analyses for particular fundamental types of input motions and combines them with the proper probabilistic concepts, one can arrive at probability distributions for the responses in the particular structure. Analyses of this type take quite sophisticated computers and relatively long times. Moreover, they do not lend themselves to the concept of a preliminary design. One can only investigate by this means a structure that has already been designed.

The second type of analysis involves the use of the response spectrum, and the combination of the responses of the various modes of a structure for which a preliminary design has already been made, with the response spectrum techniques described in Refs. 1 through 5. The basis of the modal analysis procedure is essentially the following. One can arrive at an upper bound to the stress at any point, or the value of any structural response parameter, by taking the upper bound as the sum of the absolute values of the particular response parameter for each of the modes of the structure [Ref. 10]. A better measure of the most probable value of the maximum value of that response parameter is, however, the square root of the sum of the squares of the maximum responses in each of the modes, unless the number of modes is relatively small [Ref. 11]. This is the case because of the fact that the absolute maximum values in the different modal responses occur at different times, not simultaneously. This method also requires that the structure be designed, since the method is applicable only to a structure which already exists in concept and in dimensions.

The third method consists essentially of one of several empirical procedures which involve the assumption of some type of force or acceleration distribution over the height of the structure, with some specification of the maximum value of force at a given point or plane. The method used in the Uniform Building Code specifications is of this type. These methods have the advantage that they can be used to prepare a preliminary design since they do not depend on the structure already being designed, although they may admit some inaccuracies since they can not take into account the detailed layout and characteristics of a particular structure. In general, when he uses an empirical method or a building code criterion, the intent of the designer is to choose the parameters for his design method so that the results of his analysis are in reasonable agreement with more accurate analyses of either of the two preceding types.

3. Requirements for Further Advancement of Knowledge

General Concepts

In order to increase the effectiveness of earthquake resistant design, additional information is needed, based on further research and on observation of actual occurrences. The most important need is to have better observations of the actual motions involved in strong motion earthquakes. Unfortunately, only a relatively small number of recent earthquakes have occurred in regions where there was adequate instrumentation to give complete time history records of the motions developed. Without such records, it has been difficult or almost impossible to interpret the damage observed in terms of the causes which produced such damage. In Appendix 4, Earthquake Engineering, to the as-yet unpublished "Earthquake Prediction" report to the Federal Office of Science and Technology, 1965, Drs. G. W. Housner and D. E. Hudson make the following statement:

"In recent years, a notable series of destructive earthquakes has rocked the world: Mexico (1957), Chile (1960), Agadir (1960), Iran (1962), Skopje (1963), and Alaska (1964). For not one of these earthquakes is there even one single measurement of the ground motion in the region of destruction. Only some 80 instruments are now in service in the United States. Instruments in Alaska were installed only after the 1964 earthquake and several aftershocks were recorded. The importance of coverage in areas not usually thought of as highly seismic was brought out very clearly in the recent Niigata (Japan) earthquake in 1964. Although Niigata was not located in a zone considered by Japanese seismologists to be their most active region, strong motion accelerographs had been installed there and at Akita, some 150 km away. Important accelerograph records were obtained at both sites, which have resulted in a different interpretation as to what happened than would have been arrived at without the records."

In addition to the requirements for measurement of actual strong motions in earthquakes in different regions, there is need for instrumentation of buildings to determine their response. The only building which was even moderately well instrumented and for which records were obtained in an earthquake is the Latino Americana Tower in Mexico City [Ref. 12], for which readings were obtained of relative story deflection in the first, 25th and 39th story in the major earthquake of July 28, 1957. As it happened, these measurements indicated close agreement with the predicted relative story motions, as interpreted from the predicted shearing forces in these stories, based on modal analysis of the structure for the design [Ref. 13].

Although much information can be obtained from assessment of damage and observations of failures in earthquakes, much more can be obtained if such examples can be interpreted with relation to the forces and motions which cause the damage. Of course, the major difficulty that faces the researcher is the fact that the earthquake is not predictable in occurrence, and that many more regions and structures have to be instrumented than those from which one expects to get readings in order to have any chance at all to obtain a correlation. It is essential that a coordinated program be undertaken of placement of instruments of various kinds to determine gross motions and structural responses. A second requirement is that of making available to the engineering profession the results of such observations in order that greater accuracy and economy can be achieved in earthquake resistant design.

The situation is almost as bad for the case of major wind forces. However, wind storms occur much more frequently than earthquakes, so that there is a greater opportunity for response of instrumentation during the life time of a structure when it is instrumented for wind force than when it is instrumented for earthquakes. However, in many respects, the response characteristics of the structure and the levels of response can be determined with the same type of instrumentation for both earthquake and wind.

Other topics on which further information is needed include appropriate methods of design of foundations, behavior of foundations under earthquake loading, including liquefication of the soil underneath the foundation, and similar topics, some of which are described later in more detail. Greater attention is needed to provisions for protecting people by proper selection of details and of framing, in order to avoid gross damage leading to loss of life wherever better types of construction will permit this end to be achieved. Finally, encouragement should be given to the development of completely new concepts and new types of structures. Standard building codes and design methods have been developed and are reasonably dependable for standardized framing and structural layouts. However, much more needs to be done for unusual structures of the type under development in modern architectural treatments.

Better Description of Loading or Motion Intensities

The primary input for which information is desired for earthquakes is ground motion, and for wind it is external force acting on the structure. In both of these types of loading, better description of the intensities of force or of motion are needed. For earthquake motions, specifically, the nature and interaction of the horizontal and vertical motions is required in order to permit better assessment of the behavior of buildings subjected jointly to the combined motions. Motions at or near a fault, and the effects of fault motions close to or under a building are also of interest in some types of construction. The behavior of the soil and rock under earthquake loadings, which contribute to the forces transmitted to structures built on or within the earth, are essential to a better understanding of the earthquake problem. Of particular importance is the effect of large or deep excavations under buildings for basements, the connection of utilities to buildings, and the anchorage of tall buildings to rock to prevent difficulties with overturning tendencies.

For wind forces, better descriptions of the relative intensity of wind at various levels of a building, and the local pressures and suctions around the building are needed; a reasonable understanding of these influences is now available, and model tests in wind tunnels have been and can be made to throw additional light on these matters. However, for both earthquake and wind, better interpretation of the effects of the motions generated in the building on personnel located within the building is required in order to permit better definition of the permissible design levels.

Development of Methods of Analysis

Although methods of analysis for dynamic loadings have been developed in great detail in recent years, further developments are needed including better simplified preliminary design methods that take into account more of the parameters involved in the design of the building, such as the selection of framing, the material used, the variations of mass and stiffness with height, etc. Balance must be achieved between the simplicity and the generality of such design methods. Possibly a range of procedures would be desirable, enabling one to start with a simple preliminary design, and then modify it with relatively simple techniques for the next attempted design, prior to the review of the design with a more elaborate analysis using a computer.

The next stage, of course, involves more elaborate analyses, generally using high speed digital computers, which can be employed to review the adequacy of designs in more detail for special cases. Further attention to simplification of these methods, permitting greater accuracy in regard to the assumptions made in the analysis, are required. Such methods should take into account the behavior of the joints and connections, and the appropriate levels of damping in the different modes of action of the building, the interaction of the structural framing with the non-structural components within the building, and conditions approaching failure, to insure that the mode of failure is not such as to cause calamitous or hazardous destruction and loss of life.

Correlation of analytical techniques with model tests appears to be a necessity, because of the relative impossibility of obtaining correlation with actual earthquake events. However, where fortuitous correlation is possible, the greatest use of the data obtained from the earthquake observation should be made, and correlated both with model tests and with theory, in order to permit a better interpretation of the occurrences.

Properties of Materials and Structural Elements

Although a great deal of information is available on the behavior of reinforced concrete, prestressed concrete, and precast concrete, further information is needed both on the parameters governing the properties of the materials, and on the strength and ductility of structural elements made from these materials. The influences of state of stress, rate of loading, repeated and reversed loading, temperature changes, nuclear radiation, strain aging and stress corrosion in steel reinforcement, cracking and splitting of concrete, creep and flow, shrinkage, and similar topics are of importance in all aspects of the use of concrete with reinforcement.

It is particularly important, when concrete is used to resist dynamic loads, that good information be available on bond and anchorage of reinforcement, including prestressed or post tensioned tendons, on the shearing stresses, diagonal tension cracking, necessary web reinforcement or shear reinforcement, and on the compressive strength and ductility of concrete subjected to relatively high compressive loads combined with flexure. In addition, the tensile behavior of reinforced concrete is of great interest in connection with the overturning effect on concrete buildings. Methods of resisting collapse or damage to reinforcement in the outer columns of such buildings is of prime importance especially with narrow or slender structures and towers.

Both the strength and ductility of the various structural elements used in a building are of importance in connection with earthquake resistant design. The strength can be measured by the maximum load that the member can carry for permissible amounts of deformation corresponding to the various levels of resistance which the member can mobilize. It is of interest to have accurate information on the strength under conditions which correspond to only minor cracking and which would not require repairs in a building, as well as the level of strength and deformation that can be mobilized before collapse is imminent. This information is needed for beams, including beams with high shear combined with flexure as well as high compressive stresses; for walls deforming both in their plane and transverse to their plane, acting as slabs; for slabs or floors, with special attention to the interaction with their supporting beams and girders, and with the columns and column capitals in flat slab or flat plate construction. The proportion of the width of floor that enters into action of the floor member as part of a transverse frame is required in order to define both the stiffness and the strength of the structure.

Finally, because of increasing use of arches, domes, shells, and members having curved elements, greater attention to such members in uses involving earthquake and wind loadings is needed.

Strength and Ductility of Joints and Connections

Structural elements must be connected together to form a building. Although a great deal of information is available on structural elements, and in spite of the fact that a great deal more is needed as outlined in the preceding section, very little information of a definitive nature is available on the strength and ductility of joints and connections between members, especially those between members of different types such as the connections between columns and slabs, or between walls and girders or lintel elements. It is particularly important to define rational and accurate means of reinforcement around openings in walls to avoid damaging cracks and even failure of shear walls. The particularly unattractive and damaging "X" cracking in the portions of the walls spanning between vertical shear walls in areas where windows are located are particularly noticeable in earthquakes such as the Anchorage earthquake; methods for designing such elements so that they will remain more nearly intact need development. This may require particular attention to the details of the reinforcement and development of means of allowing deformations to take place without crushing or shear cracking in the slender elements which tie stiffer elements together.

One of the major difficulties in reinforced concrete beam girder and column construction is the problem of arrangement of reinforcement at the joint or connection where all of the reinforcing bars meet. Ingenuity is needed in developing ways of putting together these elements, possibly with precast joint details which can be used in new ways to connect to the structural elements themselves, avoiding some of the practical difficulties involved in the construction of cast-in-place concrete frames.

Composite structures and particularly those involving the combination of precast and cast-in-place elements, have been used with great success in many applications. However, further attention to the types of details that can provide greater dynamic resistance is needed because many types of construction that are adequate for static strength do not appear to have the capabilities of resisting dynamic loads adequately. An important example is welded joints in reinforcing bars, often used in precast construction.

Much of the damping in a building arises from the energy absorption at joints and connections, although a great deal of damping may occur by interaction of nonstructural components or even structural components such as partitions with the main frame members. Further attention to the ways in which damping and energy absorption occur in joints and connections is also needed.

Strength and Ductility of Structural Systems

We have dealt with a number of components of the structure including the materials used, individual members made of these materials, the joints and connections between these members, and the like. However, the building is an assemblage of all of these parts. It consists of more than the individual members and their connections because it is built on a foundation or in the ground and interacts with the foundation when the latter is subjected to motions or when the building is subjected to loads. Hence, we are concerned with the entire structural system and its behavior under earthquake or wind loadings, or other lateral loadings. 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. A number of topics are mentioned only in passing. Possibly others may be defined as well, of equal importance or perhaps of even greater importance. Nevertheless, the topics described have already been identified as causing difficulties and uncertainties, and are certainly topics on which knowledge is grossly incomplete.

The interaction of the building with its foundation may lead to energy absorption, similar to damping within the members and joints, that may have a great effect on the behavior of the building. For example, under loadings less than those which cause yielding, the damping of prestressed concrete members may be as low as only 2 percent of critical, and of reinforced concrete members with moderate crack openings may be less than 4 percent. Even when joints are present and permit greater energy absorption, these values of damping are not greatly increased. They may be compared with the damping that has been observed in major structures subjected to stresses below working stress levels, of only 0.5 percent critical or less. However, the interaction of a building and its foundation may absorb energy to an even greater extent and lead to actual effective damping values of the entire assemblage of as much as 5 to 10 percent. Inadequate information is available on this point. It is of course a function of the type of foundation and the possibilities of interaction of the basement walls, floors, footings, etc., of the building with the foundation materials.

The importance of openings in shear walls and reinforcement around such openings, and of interaction between shear walls and flexural frames when these are used in a composite way in a building assemblage has already been mentioned. The strength of a building is not necessarily the sum of the strengths of its component parts even when these are designed to act together in a composite fashion. Owing to the difference in ductility of the different components of the building, the stiffer part may fail before the more flexible part may even begin to develop its strength. This type of problem arises especially in interaction between shear walls and flexural frames in a building strengthened by both types of elements. Therefore it is essential to have information concerning the resistance-deflection relations for the various kinds of elements which stiffen a building or strengthen it against lateral forces, in order that the interaction of these various elements can be evaluated.

Among additional questions that need consideration are such topics as connections to slip formed walls, including the support of girders and beams on such walls, and the reinforcement of openings in these walls; the provision of lateral bracing in lift slab structures; the question of bonded versus unbonded prestressing tendons in prestressed concrete construction; the behavior of prestressed anchorages under dynamic loadings; and the splicing of reinforcing bars, particularly large size bars and the connection of reinforcement to foundations.

It is clear that although much is known about the behavior of reinforced and prestressed concrete buildings under dynamic loadings, much more remains to be learned. It is expected that with the cooperation of the engineers from the various countries represented in this congress, many of these questions will be resolved in the near future.

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