Experiments with steel members and their connections under repeated loads

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Experiments with Steel Members and their Connections under Repeated Loads

Essais d'éléments en acier et leurs liaisons soumis à des charges répétées

Versuche mit Stahlbauteilen und deren Verbindungen unter wiederholter Belastung

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1. INTRODUCTION

For many years rational design of civil engineering structures has been based almost entirely on the assumption of the elastic behavior of materials. This approach has not lost its validity at service loads normally encountered during the life of a structure. However, the necessity for designing more economical structures and the interest in true factors of safety led to the study of the ultimate load-carrying capacity of such structures and to the development of plastic methods of design based on the concepts of limit analysis. This newer approach is based on the assumption of applying a monotonically increasing load until failure occurs; and for ductile materials, the developed theories lead to excellent estimates of the ultimate limit state for a structure under such circumstances. On the other hand, during a strong motion earthquake, a structure such as a building is subjected to random cyclic loadings that may cause inelastic behavior in members and their joints. This necessitates a study of the behavior of members and their connections under repeated loads. In comparison with the experiments in which the applied load is monotonically applied until failure, information on the behavior of structural elements under repeated loading is rather meager.

In this paper, remarks are limited to the behavior of structural elements in moment-resistant steel frames under prescribed cyclic loading. Therefore, although the earthquake problem that motivates this study is a nondeterministic one, in the work reported here the intensities of the induced cyclic load or displacement amplitudes are prescribed *a priori*. Further, in this investigation mainly the extreme conditions simulating earthquake actions are considered. Interest is centered on the ultimate limit state under cyclic loading corresponding to the maximum load-carrying capacity of steel members and their connections. In many respects, the reported experiments may be characterized as "overtesting," i.e., the members are subjected to extreme actions that normally would not be expected in practice. Nevertheless, it is important to know the ultimate limit-state capabilities of the steel members under extreme repeated loads to be knowledgeable of the true safety margin.

Some phases of the work have reached the stage of development where mathematical models can be suggested. This is done for the inelastic cyclic behavior of beams, as well as for a particular type of a panel zone in a column. No recommendations as yet are given for a model to represent a plastic hinge in a column. Other important situations that may be troublesome are also pointed out, but their behavior is described only in qualitative terms. This pertains to doubler plates and possible buckling failures.

The question of low-cycle fatigue is examined, and a tentative suggestion for analyzing such situations is made. A brief discussion of the brittle failure phenomenon encountered with steel is also brought in.

2. MOMENT-RESISTANT FRAMES

2.1 THE GENERAL PROBLEM

A typical portion of a moment-resistant steel frame consisting of a series of vertical columns and horizontal beams is shown in Fig. 1(a). A good deal of information is available on the elastic and inelastic behavior of these individual members under monotonically increasing loads. Less is known under the action of repeated and reversed cyclic loading. Some information of this type for beams will be presented below.

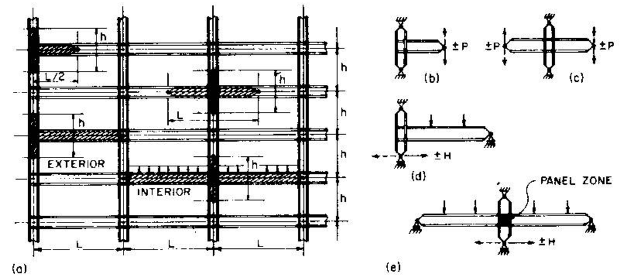


Fig. 1: Moment-Resistant Frame and Subassemblages

Moreover, since in actual construction the beam depths may be on the order of 10 per cent of the column height in a typical building and some 20 per cent or more at the lower floor levels of tall buildings designed to resist seismic forces, the highly stressed regions of the column corresponding to the beam's depth also require investigation. As experiments with complete frames are prohibitive in cost under normal circumstances, much attention by the investigators has been directed to selecting meaningful subassemblages that would provide the necessary data. Several possibilities of subassemblage designs are shown cross-hatched in Fig. 1(a) and are repeated in Figs. 1(b), (c), (d), and (e). A detailed experimental analytical study of such subassemblages can provide very useful information on the load-deformation characteristics of complete frames in elastic as well as in the inelastic range for specified cyclic loadings or displacements.

The subassemblage of the type shown in Fig. 1(b) has been used in an important study [1]. Since the horizontal displacements of the top and the bottom of the column in this type of an experiment is prevented, no Pδ effect is included in such experiments. This is also true for the specimen of the type shown in Fig. 1(c) [2]. Moreover, in both of the above specimens, no provision is made to properly account for gravity loads. Further, since moment-resistant frames are statically indeterminate, correct distribution of the applied moments in the inelastic range cannot be achieved. For this reason, although the subassemblages of the above type are useful, the results obtained from such experiments must be carefully interpreted.

An indication of how a column and a panel zone act in a frame is illustrated in Fig. 2. Note that three items contribute to the story-drift: deformation of the column, rotation of the beam, and panel-zone deformation which is principally caused by shear. It is the combination of all three of these effects that contributes to the Pδ effect. The subassemblage shown in Fig. 1(e) with the outer ends of the beams and the bottom of the column on rollers represents fairly realistically the actual conditions existing in a typical steel frame. By providing torsional restraint at the outer ends of the beams, the model would be improved; but this was not considered essential in the installation referred to later [3,4]. The subassemblage of Fig. 1(d) is similar to that of Fig. 1(e) and is useful for the study of outside columns.

The elastic and inelastic behavior of beams, panel zones, and columns under repeated loads is discussed below, with emphasis being placed on beams and their connections. These data should be sufficient to determine complete load-deformation histories under cyclic loadings of subassemblages as well as complete frames. Analytical formulation of this whole problem is not entirely complete at this time.

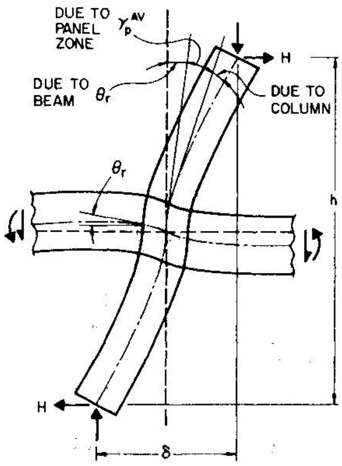


Fig. 2: Components of Story Drift

2.2 BEHAVIOR OF BEAMS

The commonly accepted approach of designing earthquake moment-resistant frames tries to avoid significant inelastic action in the columns. Further, by means of doubler plates or other types of reinforcement, the panel zone deformation is kept to a minimum. For these reasons, it is especially important to study the inelastic behavior of beams and their connections under cyclic loading.

Based on the above approach, a typical interior span with negligibly small gravity loads would have the moment diagram shown in Fig. 3(a), and the deflected shape as in Fig. 3(b). The dashed lines correspond to the moment diagram for a lateral load acting in the opposite direction. A cantilever with a cyclically applied force P at the tip, as in Fig. 3(c), provides a suitable experimental arrangement. For situations where gravity loads are also acting on the girder, the distance ℓ must be made shorter. In such cases the simulation of the actual conditions that develop in a beam is less accurate.

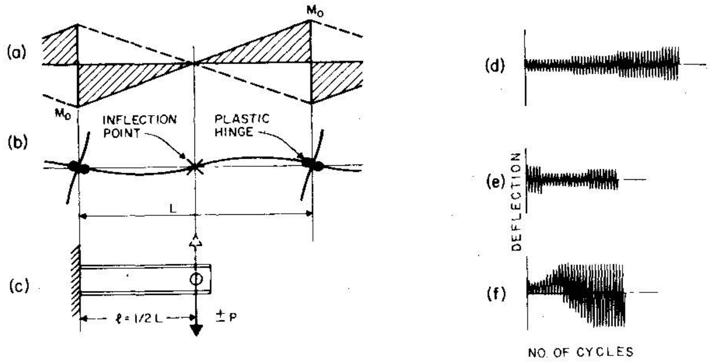


Fig. 3: Cantilever Idealization and Programs of Cycling

Experiments with many cantilevers were performed at the University of California, Berkeley [5,6] Of these, 24 were done with W 8 x 20 specimens of approximately 5 ft. in length; 5 with W 18 x 50 beams, and 3 with W 24 x 76 members. The 8 large specimens were approximately 7 ft. long. Many different types of connections were used in these investigations. These included all-welded, bolted, and hybrid types with welded flanges and bolted webs. In all cases, cyclic load applications were decided a priori and were intense enough to cause inelastic behavior in the specimens. The most frequently used scheme of loading was of the stepladder type with the induced tip deflections progressively increasing in their magnitude, Fig. 3(d). In some instances, the inelastic excursions were begun with very large amplitudes and then were returned to the stepladder type, Fig. 3(e). Displacements that randomly varied from one side to the other were used in some experiments, Fig. 3(f). These variations in the prescribed loading paths were made to determine history dependence on the results.

During the final stages of the experiments the specimens were usually severely deformed, and in all cases they either fractured or the displacements reached the capacity of the equipment. The photographs in Figs. 4, 5, and 6 show some of the specimens at the end of the experiments. The flange crack in

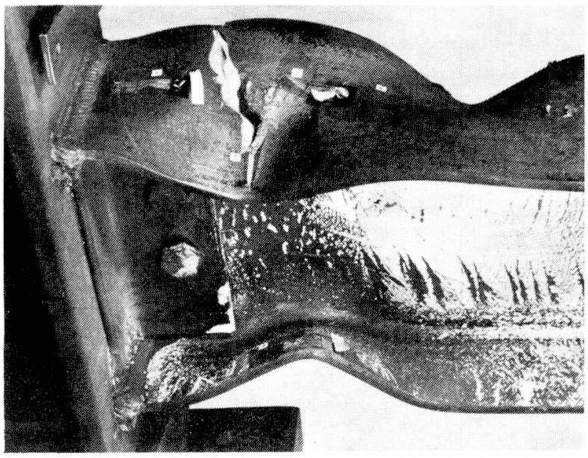


Fig. 4

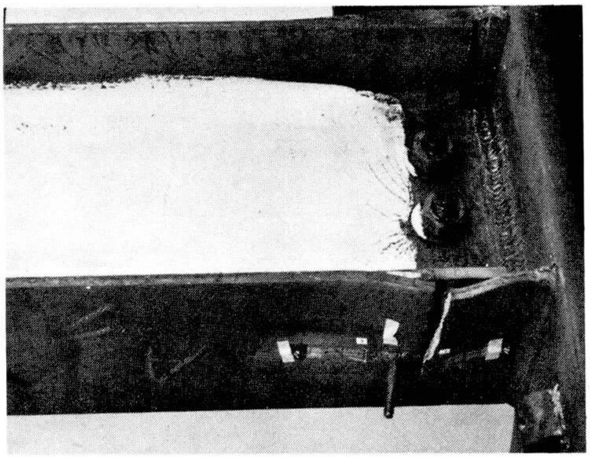


Fig. 5

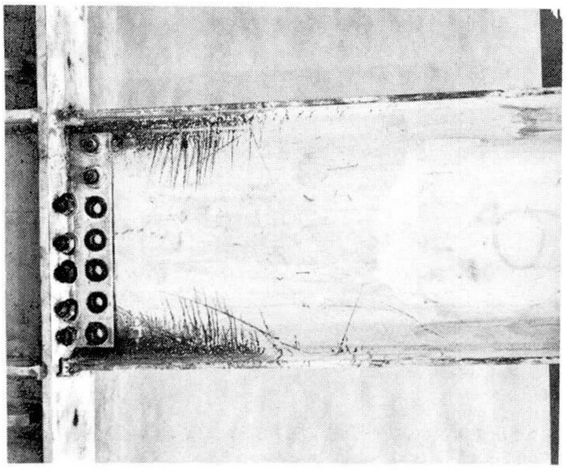


Fig. 6

a direct welded specimen, Fig. 4, began at a tackwelded stud used for instrumentation. For the specimen with connecting plates, Fig. 5, the fracture took place at the end of a fillet weld. A fractured specimen with a bolted web and welded flanges is shown in Fig. 6. Incomplete participation of the web at high loads may be noted by observing that whitewash did not flake off across the whole depth of the member.

The most important characteristic of the behavior of these cantilevers is exemplified by their load-deflection curves which, for cyclic loading, are their hysteresis loops. A good example of a series of hysteretic loops for a W 24 x 76 is shown in Fig. 7 [6]. In this experiment, the induced tip deflections were progressively increased in a manner indicated in Fig. 3(d). The similarity of the consecutive loops for the same displacement is to be noted. Since the area enclosed by the hysteretic loops indicates the amount of dissipated energy, such repetitiveness, synonymous with reliability, is an important property of structural steel members.

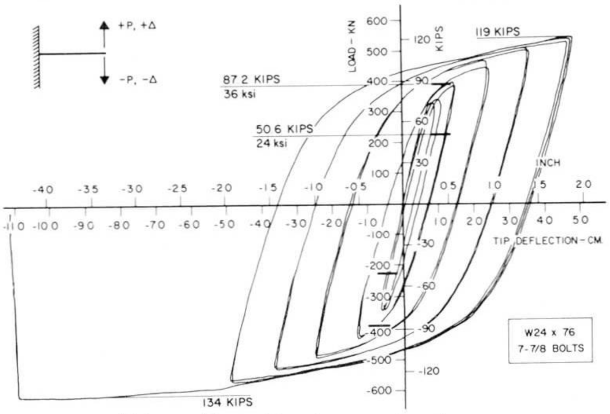


Fig. 7: Hysteresis Loops

The hysteresis loops shown in Fig. 8 [7] correspond to the cycling pattern of Fig. 3(e). The initial elastic excursions for this W 8 x 20 member were very large, and it is important to note that essentially the same shape loops are observed as in Fig. 7, although some upward drift of the loops can be observed. Nevertheless, practically speaking, a rather weak history dependence characterizes the behavior of these laterally braced steel members under cyclic loading. Load-deflection loops enlarging to the right

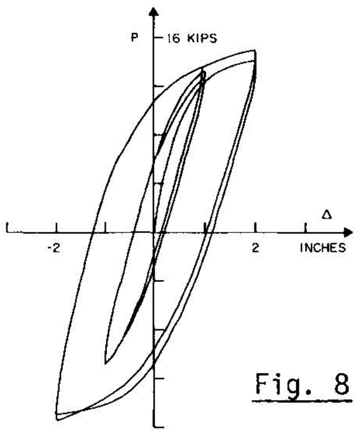


Fig. 8

SPECIMEN W2B-C10

generated by a W 8 x 20 cantilever specimen are shown in Fig. 9 [7]. Their shape remains quite similar to the ones of Fig. 7. This fact reinforces an assertion made earlier [8] that a moderate shift of the hysteresis loops along the deformation axis is possible. The above two observations provide some justification for mathematical idealization of the hysteresis curves.

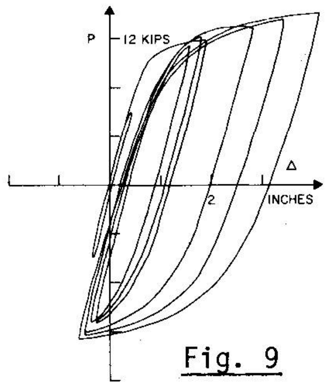
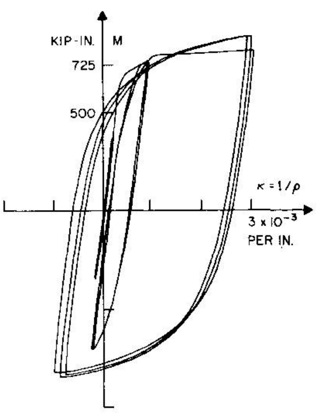


Fig. 9

SPECIMEN F1-C6

2.3 IDEALIZATIONS OF CYCLIC BEHAVIOR OF BEAMS

Hysteresis loops of the type shown in Figs. 7, 8, and 9 can be very accurately represented using a Ramberg-Osgood function augmented by Masing's hypothesis [5,9,10]. This formulation has been used in analyzing some simple frames [11]. However, these functions are not always convenient to apply; and, what is more important, the load-deflection characteristics of a cantilever beam are not the fundamental quantities for a general frame analysis. If one were assured that buckling is not critical, it would seem best to begin with a stress-strain diagram obtained for cyclic loading; then to integrate this to obtain the moment-curvature relation for a given member; and, finally, using the latter information, to calculate the load-deflection response due to cyclic loading.



SPECIMEN F1-C1

Fig. 10

It can be fully anticipated that stress-strain hysteretic curves for different steels will gradually become more generally available, and the procedure stated above could be followed. For the present, moment-curvature relationship can be easily obtained from any cantilever experiment. By measuring longitudinal strain near the support with an electric strain gage and by neglecting local buckling, the curvature of the member becomes known. Since the location of the gage is also known, the bending moment corresponding to a given curvature is established. Using this procedure the moment-curvature diagram shown in Fig. 10 was constructed. Note that except for the first new and different excursion, the curves in each group repeat themselves. Again an accurate representation using a Ramberg-Osgood function can be achieved, but this does not seem to be necessary.

For cyclically repeating loads, the skeleton curve for the moment-curvature relationship M-ϕ may be approximated with a sufficient degree of accuracy for many purposes by a simple bilinear relationship shown in Fig. 11. On this basis, the agreement between calculated and experimental results for P-Δ is quite satisfactory. If a better agreement is desired, the use of a trilinear approximation for the moment-curvature relationship improves the results, Fig. 12. (As before, the calculated results are shown with dashed lines.) Analogous conclusions were reached by another investigator [16]. To obtain accurate results for the first application of a large load, the established procedures of elastic-plastic analysis should not be abandoned.

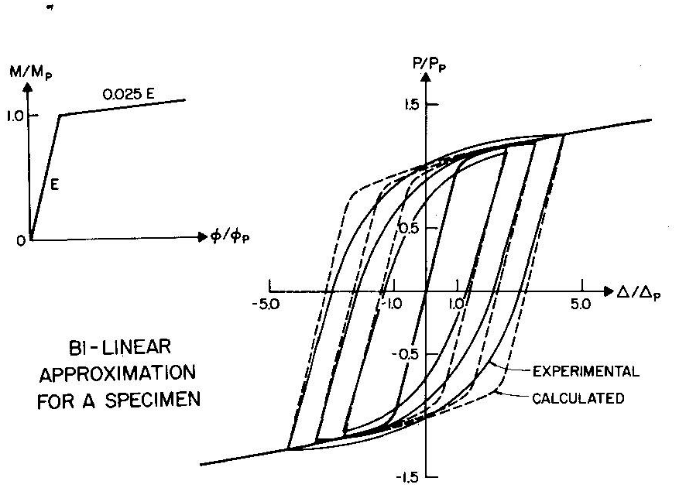


Fig. 11: Load-deflection Hysteresis Loops

The bi- or trilinear moment-curvature relationships appear to be both simple and sufficiently accurate for most practical applications in dealing with cyclic loadings. If properly used, they should provide a good indication of hysteretic damping in steel structures; however they are not universally applicable to all steel members regardless of the connection used. For example, load-tip deflection curves for a bolted connection are shown in Fig. 13 [7,8]. These hysteresis loops do not resemble the ones discussed previously. Initially, the high-strength bolts held well--on the first load reversal their behavior is still very good--but then the deteriorated but unique hysteresis loops persist for many cycles. In these later stages of cycling one notes substantial regions of slip and of joint stiffening once the bolts become seated.

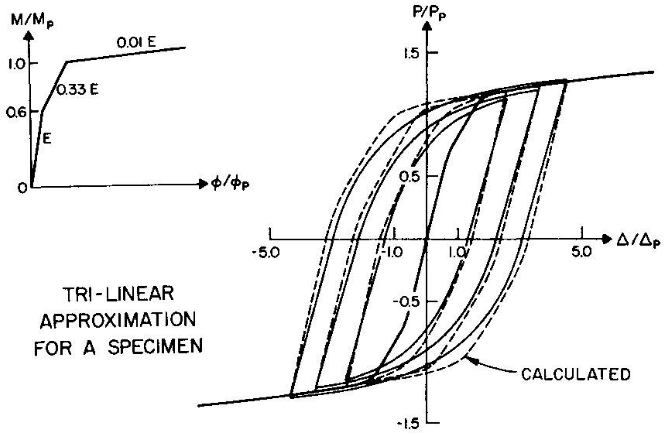


Fig. 12: Hysteresis Loops

Another anomalous situation is shown in Fig. 14, where the connecting plate of the type shown in Fig. 5 buckles and thereby temporarily reduces the stiffness of the joint.

2.4 BEHAVIOR OF PANEL ZONES

The deformation of the panel zone was studied using subassemblages of the type shown in Fig. 1(c) [4,12,13]. As may be noted from Fig. 2, the horizontal force H acting with a lever arm h and the vertical force P (not shown) with an arm δ apply axial forces, moments, and shears to the column joint. This system of forces is resisted by the forces in the beams. For this connection, the differences in beam moments ΔM at the joint can be related to the average angle of shear deformation γ_p^{av} in the panels. This angle as well as the angular rotation Θ_r of the joint due to bending of the beams are indicated in Fig. 2.

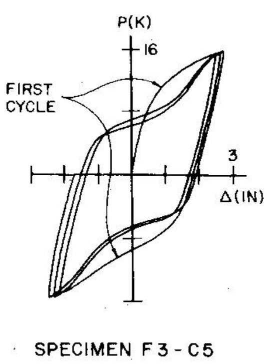


Fig. 13

The panel-zone shear deformations versus the difference in beam-end moments for a particular experiment are plotted in Fig. 15. It is important to note that these load-deformation curves for cyclic loading become the hysteresis loops. Therefore, in the inelastic range of the joint behavior, panel zones dissipate energy.

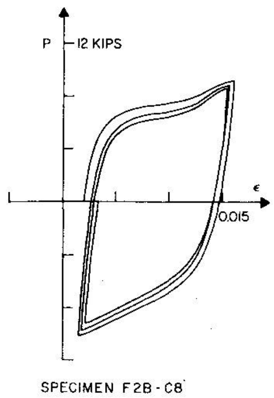


Fig. 14

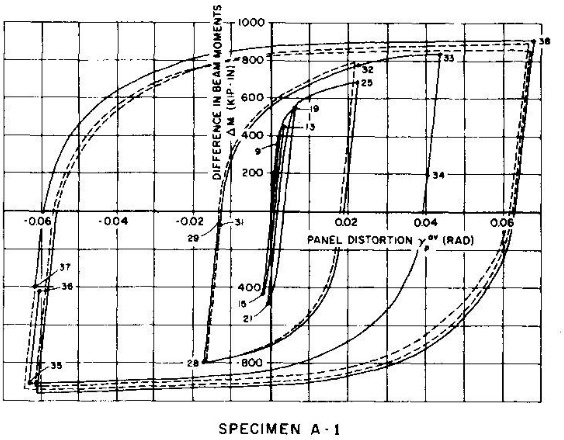


Fig. 15: Shear Hysteresis

The deformation of the panel zone contributes to story drift (see Fig. 2). This may be noted by studying Fig. 16, where the behavior of a subassemblage with a weak panel zone is shown. The experimental curve lies far below the prediction based on the conventional rigid-plastic analysis even including the Pδ effect. The experimental and theoretical results can be brought into good agreement, however, by considering panel-zone deformation. First, the experimental curve can be raised vertically by the amount Pδ/h, which is very nearly the exact equivalent of the horizontal force H caused by the Pδ effect. Then the new curve can be shifted horizontally to the left by δ_p, which is obtained by multiplying the angle of panel shear distortion γ_p^{av} by the clear column height. In this manner, excellent agreement with the calculated results is obtained. In applications, the process can be reversed leading to good estimates of story drifts. In some situations, strain hardening of the material must also be included.

The illustrated case is rather extreme, and in most practical cases the story drift caused by the panel distortion is considerably smaller. However, it is important to point out that the current procedure [14] for designing the panel zones, while providing sufficient strength to resist the difference in beam end-moments, gives inconsistent results for deformations in the inelastic range. An improved approach is suggested in Ref. [13].

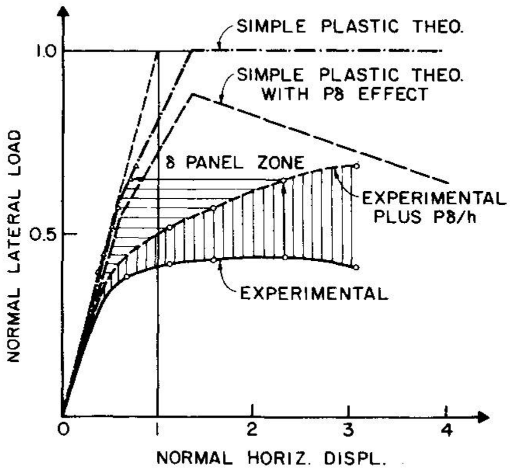


Fig. 16

In an attempt to minimize story drift due to panel-zone deformation, doubler plates are frequently used. They are quite effective, but experimental results indicate that shear is not distributed directly in relation to the plate thicknesses of the web plates. The column web takes up a substantially larger part of the shear than do the doubler plates. Thus for stiffness, the doubler plates cannot be considered to be as effective as the column web. An example of this situation is shown in Fig. 17. Similar conclusions were reached by other investigators [15].

2.5 PLASTIC HINGES IN COLUMNS

As stated in Article 2.2, the commonly accepted approach for design of earthquake moment-resistant frames tries to avoid significant inelastic action in the columns. However, considering the uncertainties of such design, some inelastic action is likely to occur in the columns during a strong-motion earthquake. Therefore, it is very important to know how axially loaded members behave under cyclic loading in the inelastic range.

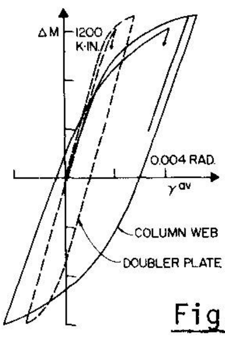


Fig. 17

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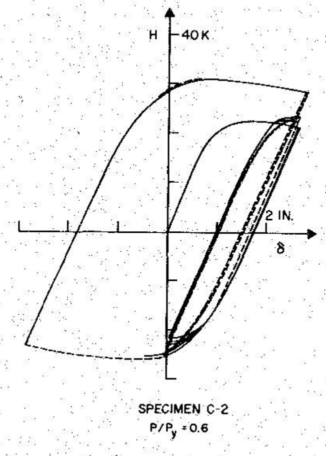

Fig. 18

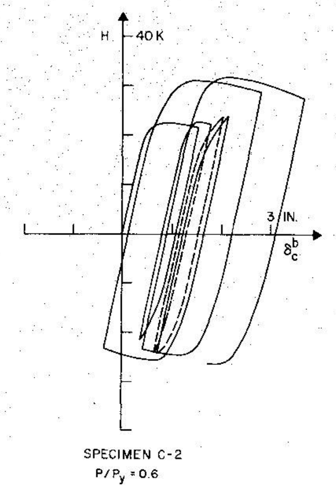

Fig. 19

With the above in mind, six experiments employing subassemblages of the type shown in Fig. 1(e) were performed. In four cases, the columns were bent around their strong axes and in two, around their weak axes. In all cases, the beams framing into the columns were selected to be sufficiently large to act elastically under the applied loadings. Therefore, the plastic hinges were forced to occur in the columns. The ratios of the axially applied loads P to the axial load P_y at yield varied from 0.8 to 0.3. When such a ratio is zero, one has the case of a cyclically loaded beam discussed previously. Thus, a very wide range of P/P_y ratios has been explored. In all of these experiments large story drifts were imposed.

The results for a typical case, corresponding to $P/P_y = 0.6$ for a W 8 x 48 column, are shown in Figs. 18 and 19. The H-δ hysteresis loops in Fig. 18 look remarkably normal and resemble the ones obtained for beams; however, this is not the complete story. Axially loaded columns when subjected to very severe cyclic loading tend to assume single curvature from top to bottom. The permanent set is such that the columns become and remain C-shaped. As inelastic strains occur, the material strain hardens and becomes stronger elastically. Therefore, on a return stroke the column does not fully straighten out, and progressively the single curvature shape becomes more and more pronounced. The displacement of the panel zone relative to both of the column ends moves in the same direction but by different amounts. The control load-displacement path is shown in Fig. 18. The resulting load-displacement path between the panel zone and the bottom of the column as seen from Fig. 19 looks quite different. As to be expected, at high lateral loads the column flanges buckle.

Essentially, the same type of behavior was observed for bending of columns around their weak axes. Some suggestions for mathematically modeling the inelastic behavior of columns under cyclic loading has already been made [16]. These results, however, were obtained on simplified small models, and further verification of the conclusions seems desirable. For the present, it seems encouraging that there are strong indications that some plastic activity in columns can be tolerated during cyclic loading.

3. LOW-CYCLE FATIGUE AND BRITTLE FAILURE

Very little, if any, evidence is available on failures of buildings due to low-cycle fatigue. Nevertheless, with ever-changing design requirements of structures, it is important to consider this problem [17]. Brittle failure of steel is also a possibility for large weldments and deserves most careful scrutiny for this reason [20,21].

To establish a damage criterion for low-cycle fatigue, the critical section of a member must be considered. The behavior of a member as a whole is not the real issue. Fortunately, however, as indicated in Article 2.3, if buckling effects are small, the general deformational behavior of a structure may be related to its moment curvature or, if necessary, to the stresses and strains at any section. Therefore, it is possible to apply the cumulative damage criterion at the point of the maximum cyclic stress or strain.

There are three possible alternative cumulative damage criteria, which may be written in the following convenient form:

Miner's Rule
$$\sum_{i=1}^{k} \frac{n_i}{N_i} = 1 \qquad (1)$$

Manson-Coffin Hypothesis
$$\sum_{i=1}^{k} \left(\frac{\Delta\varepsilon_{pi}}{\varepsilon_p}\right)^a = 1 \qquad (2)$$

Dissipated Energy Hypothesis
$$\sum_{i=1}^{k} \left(\frac{\Delta e_i}{e}\right)^b = 1 \qquad (3)$$

In Eq. 1, n_i is the number of design cycles at a given stress, and N_i is the corresponding number of cycles causing failure. In applying this relation to randomly applied cycling, an experimentally determined stress-cycle diagram with N_i's corresponding to the number of cycles (life) at the various stress levels S must be known. Then, if in a particular case the indicated sum on the left-hand side of the equation is less than unity, it is presumed that the member will not fail in fatigue. This equation is widely used by mechanical engineers [18] for stresses at essentially elastic levels and appears to be reasonably accurate for high-cycle fatigue.

For low-cycle fatigue the criterion based on plastic strain is more appropriate (Eq. 2). In this equation ε_{pi} is the plastic amplitude range (permanent deformation per half-cycle) and ε_p and a are experimentally determined constants [19]. Good correlation with experimental results can be achieved using this equation.

Another logical possibility for a cumulative damage criterion for low-cycle fatigue is expressed by Eq. 3 where Δe_i is the dissipated energy per half-cycle (full loop can be used for completely skew-symmetric cases), and e and b are appropriate experimental constants. If a skew-symmetric bilinear skeleton curve idealization is used, this equation reverts to Eq. 2. On the other hand, it is believed that this formulation is more universal and is applicable to a larger class of problems. For example, for reinforced concrete members the hysteresis loops are quire irregular, yet this approach would seem to be appropriate. Efforts to verify the above cumulative damage criterion both for steel and reinforced concrete are now being pursued by the writer and his associates.

It is believed that for steel beams, low-cycle fatigue is not a serious problem, but it would be well to have an analytical procedure to verify this fact. Eq. 2 and/or 3 provide this possibility. However, the more pressing problem in structural steel fabrication is brittle fracture. As the thickness of the members increases, and welding is becoming the conventional manner of interconnecting the members, these difficulties increase. The necessity of being appraised on such matters as the Charpy V-notch impact values, as well as the effects of temperature, loading rate, and plate thickness must not be overlooked [20]. The question of the lack of isotropy of rolled steels also is becoming increasingly important [21]. In all of this, fracture mechanics related to crack propagation plays the dominant role [20].

4. BUCKLING OF STEEL MEMBERS

The behavior of steel members described above can terminate abruptly if excessive buckling occurs. For monotonically applied loading, ratios of flange widths b to their thicknesses t, as well as beam depths d to their thicknesses t_w, are carefully limited [14]. This avoids, or at least minimizes, local buckling of the members. For cyclic loading, no similar provisions exist. From the experience gained from the experiments described here, it is clear that the requirements for repeated loading are more severe. As loads are cycled, the buckled regions tend to enlarge. Therefore, it is prudent to be more conservative in assigning maximum b/t and d/t_w ratios for cyclic loadings than for those that are monotonically applied.

Lateral torsional buckling of beams also needs to be very carefully guarded against. The bottom flange of a beam can be in compression over a considerable portion of a span. In contrast to the top flanges held by the floor system, often the bottom flanges are not laterally braced. Under cyclic loading, lateral deflections tend to magnify, and it is imperative to prevent this by bracing. Observations in the laboratory demonstrated that deep beams with unbraced bottom flanges are particularly vulnerable to this phenomenon which is very dangerous [22].

5. ACKNOWLEDGEMENTS

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