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Performance of Steel Beams and Their Connections to Columns During Severe Cyclic Loading

Comportement de poutres en acier et de leur assemblage sur colonnes sous d'importantes charges périodiques

Das Verhalten von Stahlträgern und ihren Anschlüssen an Stützen unter schweren zyklischen Belastungen

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Introduction

In the Preliminary Publication for the 1968 IABSE Eighth Congress, D. Sfintesco makes reference to some of the earlier work at the University of California on steel beam-to-column connections subjected to repeated and reversed loading (1, 2). It is the purpose of this discussion to call attention to further published results (3, 4) and to provide the readers with a summary on the new work which should shortly become generally available in print (5, 6). It is gratifying that the earlier analyses as well as the penetrating opinions and observations by D. Sfintesco expressed in the Preliminary Publication are in essential agreement with the later findings.

Conventional stationary structures such as buildings, bridges and towers are not immune to dynamic loadings. As pointed out by J. Ferry Borges in the Preliminary Publication, such loadings are associated with wind and earthquake, as well as machinery, traffic, and blast loads. The general effect on stationary structures due to such loads is essentially analogous. However, the loading of buildings caused by strong earthquakes is particularly severe.

During an earthquake the soil on which a building is situated becomes subjected to a rapid back-and-forth motion in horizontal and vertical directions. The horizontal motion usually causes the more damaging effect. A representative accelerogram for a horizontal movement for an earthquake (7) is shown in Fig. 1. The typical rapidly varying inputs at the ground level reflect themselves in relatively slow swaying motions of buildings. This is due to the fortunate lack of resonance between the frequency of the ground motion input and the natural period of vibration of typical high-rise buildings.

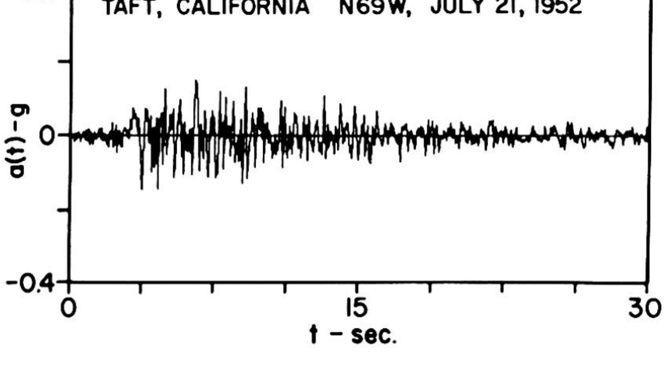
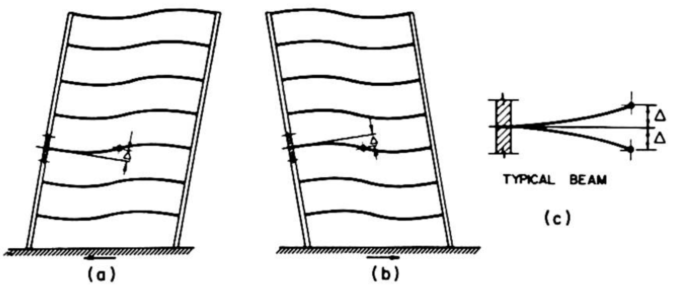


Fig. 1

The swaying motion of a severely strained building is shown schematically in Figs. 2(a) and (b), where it is assumed that the columns are relatively inflexible. This is in conformity with the current design practice which permits essentially only elastic action in the column, but allows substantial inelastic deformation in the beams and their connections to the columns.



BEHAVIOR OF A FRAME IN AN EARTHQUAKE

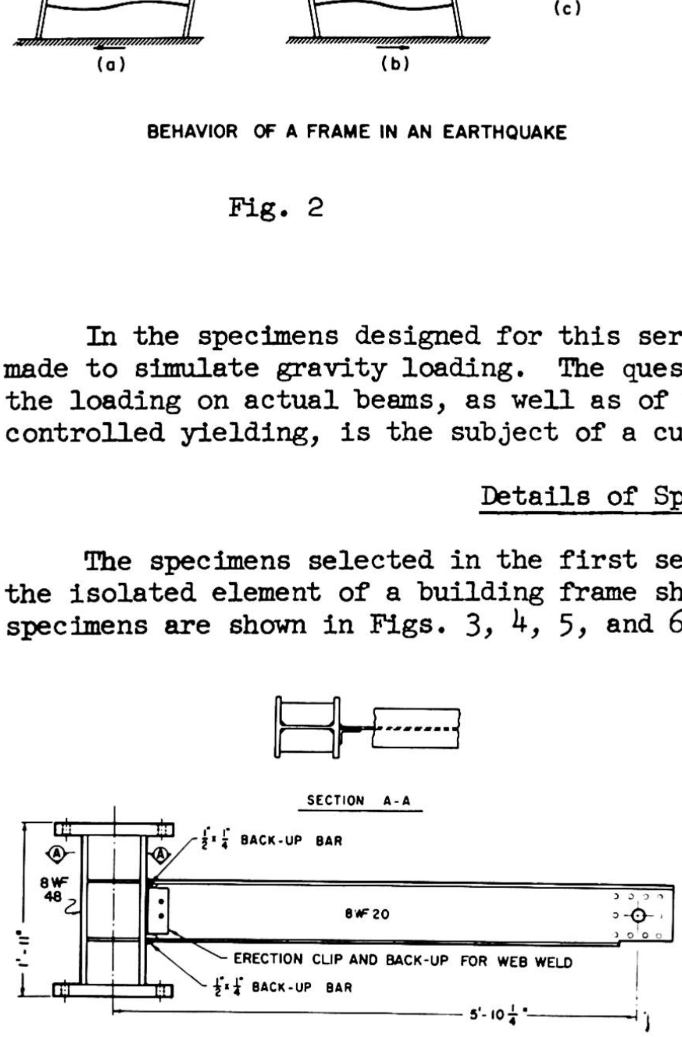
Fig. 2

On the above basis all specimens for the first series of the California experiments were so designed that yielding occurred in the beams and their connections and not in the columns. Such yielding of the members provides damping of the structure and assures dissipation of the energy input due to an external cause such as an earthquake.

In the specimens designed for this series of experiments no attempt was made to simulate gravity loading. The question of simulating more accurately the loading on actual beams, as well as of permitting columns to exhibit some controlled yielding, is the subject of a current investigation (8).

Details of Specimens

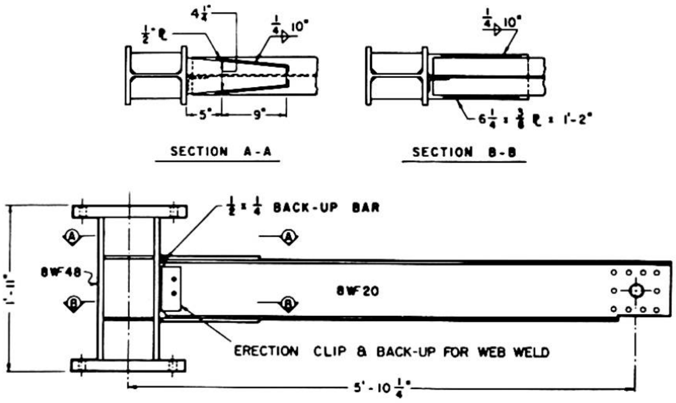
The specimens selected in the first series of the California tests resembled the isolated element of a building frame shown in Fig. 2(c). The details of the specimens are shown in Figs. 3, 4, 5, and 6. In the specimens of the F1 type, Fig. 3, the beam was directly welded to the column stub. In the specimen of the F2 type, Fig. 4, welded connecting plates were used. In the specimen of the F3 type, Fig. 5, the attachment of the beam to the column stub through the connecting plates was achieved using high-strength bolts. The welded detail W1 for connecting a beam to the web of the column is shown in Fig. 6. All of these details represent the types widely used in the construction of steel buildings.



TYPE F1 WELDED BEAM-TO-COLUMN CONNECTION

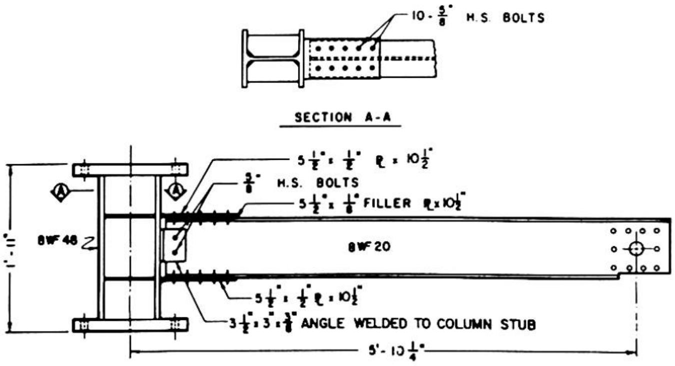
Fig. 3

In several of the specimens of the F2 and the F3 types the thickness of the connecting plates was varied. For two web connected specimens, designated as W2, the connecting plates were tapered or shaped to provide a more gradual change in the cross-section of the beam flange. A total of twenty specimens using A-36 steel were fabricated according to above details. Four additional specimens, two of the F1 type and two of the F2 type, using A-441 steel formed a part of the same series of experiments (4, 5). The specimens made of a higher strength steel are identified by letters HS and referred to as F1HS and F2HS.



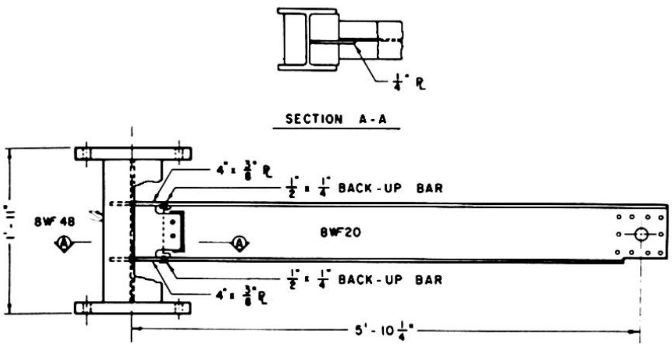
TYPE F2 WELDED BEAM-TO-COLUMN CONNECTION

Fig. 4



TYPE F3 H.S.BOLTED BEAM-TO-COLUMN CONNECTION

Fig. 5



TYPE W1 WELDED BEAM-TO-COLUMN CONNECTION

Fig. 6

Experimental Procedure

The basic experimental set-up is shown in the photograph of Fig. 7. A double acting hydraulic cylinder provided the desired load input at the tip of the cantilever. Experiments were controlled either by a strain gage near the built-in end of the cantilever, or by a selected tip deflection. Some typical loading programs are shown in Fig. 8. A considerable variety of such programs was used in the experiments.

In Fig. 8(a) the step-ladder type of sequence for the tip deflection is shown. Here the displacement amplitudes are arbitarily increased gradually. An experiment with a few strong initial displacements followed by the step-ladder sequence of the tip deflection is shown in Fig. 8(b).

Numerous measurements were recorded during the experiments (4, 5). Among these the load-deflection characteristics of the beam are particularly important.

Principal Experimental Results

Applying repeated and reversed loading of the type shown in Fig. 8 causes considerable yielding in the specimens during each cycle of loading application. Therefore, as is to be expected, after a number of cycles the specimens fail. The manner of failure is strongly dependent on the type of specimen, whereas the number of cycles to failure depends on the amplitudes of the tip deflection. These results are summarized in Table 1. For a more complete description of the experiments and the results, the reader is referred to References 4 and 5.

TABLE I

Specimen	Cycles to Failure	Type of Cycling Number of Cycles at Given Tip Deflection*	Total Energy Absorption kip - in.
F1-C1	28	5 at ± 1 in.; 5 at ± 2 in.; 10 at ± 3 in.; 8 at ± 4 in.	-------
F1-C2	22 1/2	22½ at ± 3 in.	2,411
F1-C3	120	100 at ± 1 in.; 20 at ± 3 in.	3,734
F1-C4	39 1/2	20 at ± 2 in.; 19 1/2 at ± 3 in.	2,837
F1-C6	32	5 at ± 3/4 in.; 5 at ± 1 1/2 in.; 10 at ± 1 1/2 in. to ± 4 in.; 12 at ± 4 in.	2,574
F2-C1	18	5 at ± 1 in.; 5 at ± 1 1/2 in.; 8 at ± 3 in.	-------
F2-C4	44	42 at ± 1 1/2 in.; 2 at ± 2 in.	2,495
F2A-C7	38 1/2	15 at ± 3/4 in.; 15 at 1 1/4 in.; 8 1/2 at ± 1 3/4 in.	1,054
F2B-C8	32 1/2	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 2 1/2 at ± 1 3/4 in.	533
F3-C1	9 1/2	5 at ± 2 1/2 in.; 4 1/2 at ± 4 in.	-------
F3-C5	30	30 at approximately ± 2 1/2 in.	1,533
F3A-C7	65 1/2	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 15 at ± 1 3/4 in.; 15 at ± 2 1/4 in.; 5 1/2 at ± 3 in.	2,488
F3B-C7	33 1/2	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 3 1/2 at ± 1 3/4 in.	704
W1-C7 **	37	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 7 at ± 2 in.	926
W1-C9	51 1/2	2 at ± 1 3/4 in.; 15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 15 at ± 1 3/4 in.; 4 1/2 at ± 2 1/4 in.	1,500
W2A-C7	46 1/2	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 15 at ± 1 3/4 in.; 1 1/2 at ± 2 1/4 in.	1,189
W2B-C10	30	5 at ± 1 3/4 in.; 15 at ± 3/4 in.; 10 at ± 1 1/4 in.	651
F1HS-C7	74	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 15 at ± 1 3/4 in.; 15 at ± 2 1/4 in.; 14 at ± 2 3/4 in.	3,597
F1HS-C11	73	5 at ± 2 1/4 in.; 15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 15 at ± 1 3/4 in.; 15 at ± 2 1/4 in.; 8 at ± 2 3/4 in.	3,539
F2HS-C7	35 1/2	15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 5 1/2 at ± 1 3/4 in.	897
F2HS-C9	54 1/2	2 at ± 1 3/4 in.; 15 at ± 3/4 in.; 15 at ± 1 1/4 in.; 15 at ± 1 3/4 in.; 7 1/2 at ± 2 1/4 in.	2,149

* Tip deflections are approximate, and are measured from mean position.

** Results from two defectively fabricated W1 specimens are not included.

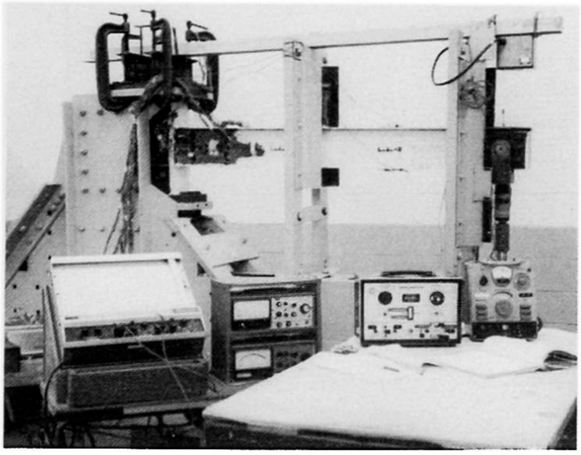


Fig. 7

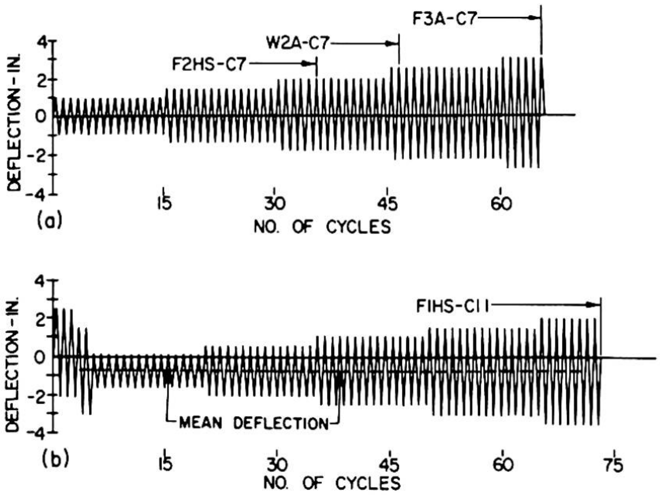


Fig. 8

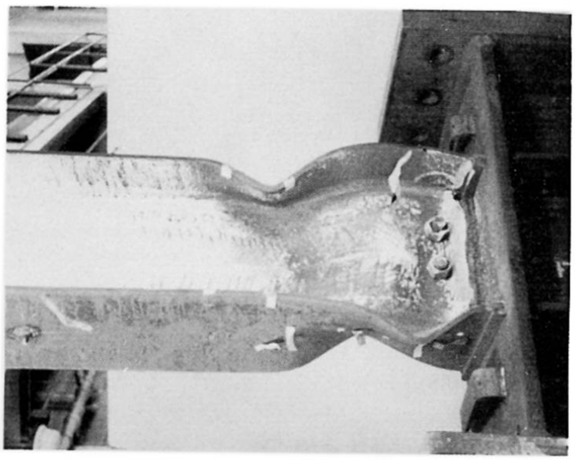


Fig. 9

A few photographs showing the manner of the ultimate failure of specimens are reproduced in Fig. 9, 10 and 11. Although such information is significant, one does not design for this condition to occur, and it is more important to answer the following two questions:

1. Can steel beams and their connections to columns withstand a sufficient number of cycles of large amplitude, i.e. of load reversals causing severe plastic strains, without breaking during a major earthquake?

2. How dependable is the energy absorption capacity per cycle during severe straining of steel?

An examination of Table 1, bearing in mind the exceptional severity of the imposed strains in the reported experiments, provides an affirmative answer to the first question. With the exception of two defectively fabricated specimens, for each specimen the number of cycles before failure occured was quite large in relation to what might be anticipated during a severe earthquake.

An examination of the hysteresis loops is necessary to answer the second question. Three sequences of hysteretic loops from one of the experiments are shown in Fig. 12. Their remarkable repetitivness and reproductibility during a number of consecutive identical cycles is noteworthy. Experimental evidence also clearly demonstrates that the onset of flange buckling does not cause the capacity of the beam to deteriorate significantly. The gradual work-softening which may be noted from Fig. 12 appears to be of no importance in seismic design as it occurs only after an excessively large number of load reversals.

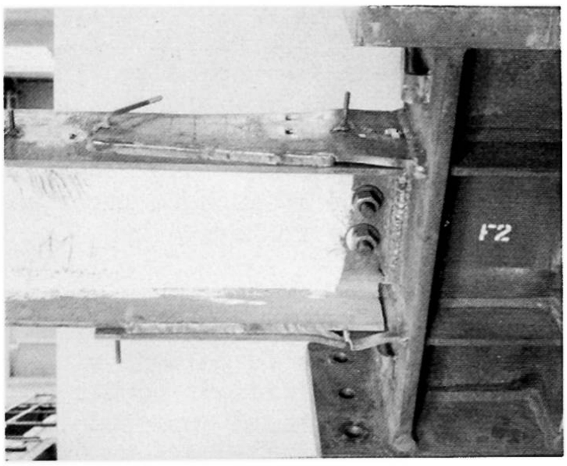


Fig. 10

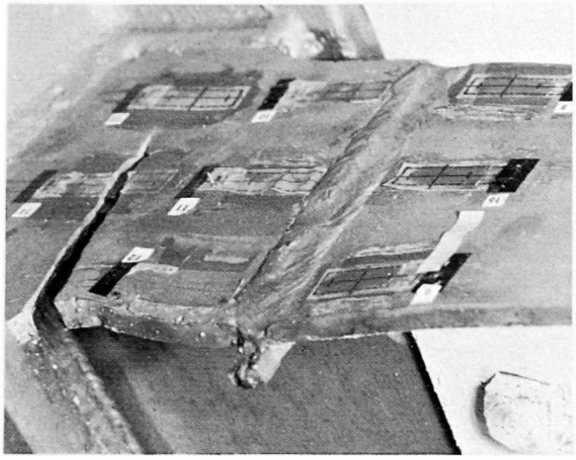


Fig. 11

The completed experiments further provided good evidence on the increase in the size of the hysteresis loops with increasing deflections. This is illustrated in Fig. 13. An approximate linear relationship between the area enclosed by the hysteresis loops with the increasing plasticity ratio has been proposed (4, 5). Therefore, on the basis of the experimental evidence, it appears that the reliability of the energy absorption capacity of properly designed and fabricated structural steel beams and their connection, is assured.

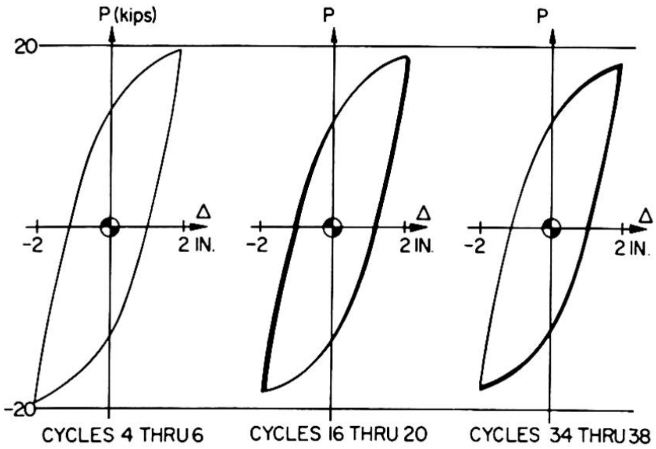


Fig. 12 Specimen F2-C4

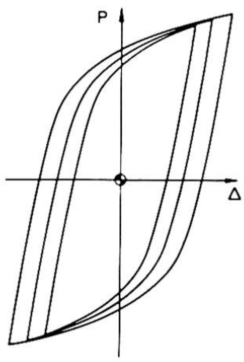


Fig. 13

Application of Experimental Results

Once one is satisfied that the hysteresis loops are reproducible during consecutive identical cycles of load application, and that the increase of the loops with increasing deflections is reasonably well established, this information can be put in mathematical form suitable for the analysis of structures. The well-known Ramberg-Osgood representation of non-linear load-displacement relationships together with Masing's hypothesis provide suitable mathematical formulations (5, 9).

Precisely this type of idealization has been applied by Berg (10) to some of the hysteresis loops generated in these experiments. By using such a formulation he studied the response of the assumed structure to a very severe ground motion are shown in Fig. 14(a), (b) and (c). The same results superposed on the same graph are shown in Fig. 15. The lateral displacements for the assumed structure having one degree of freedom are shown in Fig. 16 for a longer period of time. From this figure it is seen that often the displacements are not as severe as shown in Fig. 15 and are essentially elastic in their character.

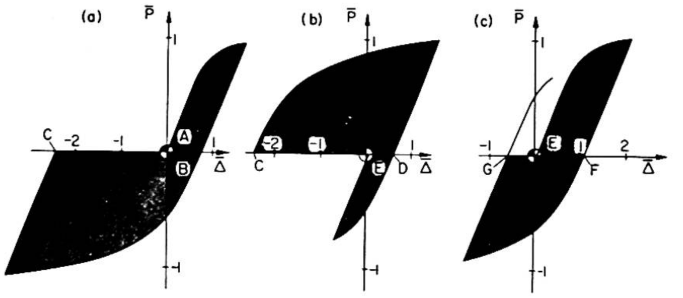


Fig. 14

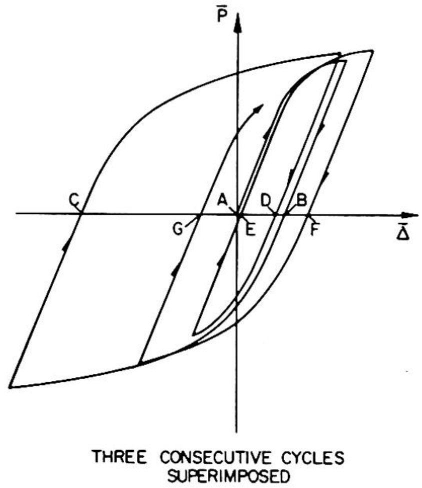


Fig. 15

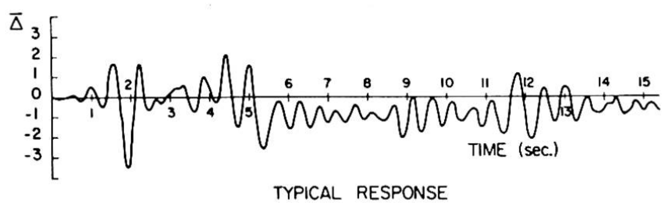


Fig. 16

Studies such as made by Berg for the above simple problem are the kind that are needed for real structures. From such studies the behavior of any one connection can be determined for an assumed earthquake. The amount of energy to be dissipated (shaded areas in Fig. 14) could be found and an assessment of the adequacy of the joint be made. Such a procedure would place the aseismic design of buildings on a more rational basis.

Acknowledgements

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SUMMARY

In this discussion attention is called to the availability of some of the new experimental results on the inelastic behavior of steel cantilever beams and their connections to column stubs during repeated and reversed cyclic loading. The described experiments attempt to simulate the conditions which develop in structural steel frames during an earthquake.

RÉSUMÉ

Cette contribution signale l'existence de nouveaux résultats expérimentaux concernant le comportement inélastique de poutres consoles en acier et leur liaison à des colonnes courtes soumises à des charges périodiques répétées et alternées. Le but des expériences décrites est de simuler les conditions qui se développent dans des ossatures en acier pendant un tremblement de terre.

ZUSAMMENFASSUNG

In diesem Beitrag soll auf einige der neueren experimentellen Ergebnisse über das inelastische Verhalten von Kragträgern aus Stahl aufmerksam gemacht werden, sowie ihren Anschlüssen an Stützenabschnitte, wenn die Träger wiederholten, zyklischen Wechselbelastungen ausgesetzt sind. Anhand der angeführten Versuche wird versucht, die Bedingungen nachzuahmen, die während eines Erdbebens in Rahmenkonstruktionen aus Stahl auftreten.

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