**Zeitschrift:** IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

**Band:** 13 (1973)

**Artikel:** Failure of steel beams due to lateral buckling under repeated loads

**Autor:** Takanashi, Koichi / Udagawa, Kuniaki / Tanaka, Hisashi

**DOI:** https://doi.org/10.5169/seals-13763

III

# Failure of Steel Beams due to Lateral Buckling under Repeated Loads

Rupture de poutres en acier due au renversement sous charges répétées

Versagen von Stahlbalken infolge horizontaler Beulung unter wiederholter Belastung


Koichi TAKANASHI
Associate Professor

Kuniaki UDAGAWA
Research Associate

Hisashi TANAKA
Professor

Institute of Industrial Science, University of Tokyo, Japan


## 1. INTRODUCTION

As pointed out by the authors of the introductory report, very few researches on the lateral buckling problems of steel members under cyclically repeated loadings can be found in the literatures. This is a report of lateral buckling tests under cyclically repeated loadings which were conducted on the steel beams with H-shaped section. These tests showed the quite different behaviors of lateral buckling as compared with the behaviors for monotonic loading tests which have been carried out by many investigators for the establishment of " plastic design " of steel structures.

Many tall buildings are recently designed on the criterion that frame structures behave inelastically during severe earthquake. Hence, a nonlinear dynamic analysis is usually performed on the assumption that structural members are expected to show stable hysteresis loops in force-deformation relationships, but the failure due to buckling and deterioration of hysteresis curves are not taken into account. In inelastic or plastic range, steel structures often fail due to buckling because of the sudden reduction of rigidity and hence the studies on the lateral buckling of members under repeated loads must be required as well as other buckling problems.

To obtain the sufficient ductility of structures, the proportions of sections and spacings of lateral supports of members must be determined to possess much plastic deformation capacity. When a tall building, which has stronger and more rigid columns than beams, is subjected to severe earthquake, " plastic hinges " develope in the ends of beams and rotate repeatedly and reversedly. Popov and Pinkney have carried out the cyclic bending tests on the cantilever steel beams and concluded that the load-deflection hysteresis loops keep remarkably stable shapes and the onset of local flange buckling does not imply an immediate loss of load carrying capacity, when closedly braced compact members are used(1,2). The beams in the real structures, however, cannot be always braced closedly even though the attached slabs provide additional and uncertain restraints, so that this series of experiments is mainly intended to examine the shapes of load-deflection hysteresis loops and to obtain the critical end rotations of beams beyond which beams will fail due to lateral buckling.

## 2. SPECIMENS AND EXPERIMENTS

Test specimens were fabricated from rolled steel beams which have H-shaped sections with depth of 200 mm and width of 100 mm. The thicknesses of flange and web are 8 mm and 5.5 mm, respectively. The slenderness ratio with respect to weak axis, $l/r_y$, for each specimen is shown in Table 1, where $l$ and $r_y$ indicate unsupported length and radius of gyration with respect to weak axis, respectively. The unsupported lengths were determined so that the slenderness ratios vary from 50 to 80, because the specifications(3,4) recommend 65 as the critical slenderness ratio. The specimens of DG series and DGH series were made of JIS SS41 steel ($\sigma_y = 2.9 t/cm^2$) and JIS SM50 steel ($\sigma_y = 4.5 t/cm^2$), respectively. The average yield stresses, $\sigma_y$, were evaluated by tension coupon tests. The full-plastic moment, Mp, of each beam was calculated for these values.

The rig employed to subject the beam specimens for reversed loadings is illustrated in Fig. 1. Reversed loads were repeatedly applied by the hydraulic actuator at the center of the span, where a guide was provided to prevent lateral displacement and rotation with respect to beam axis. At the both supported ends only lateral displacement and rotation with respect to beam axis were also prevented. This loading arrangement was intented to simulate the beam behaviors of a structure under lateral loads. The motion of the top of actuator was controlled by the servo controller.

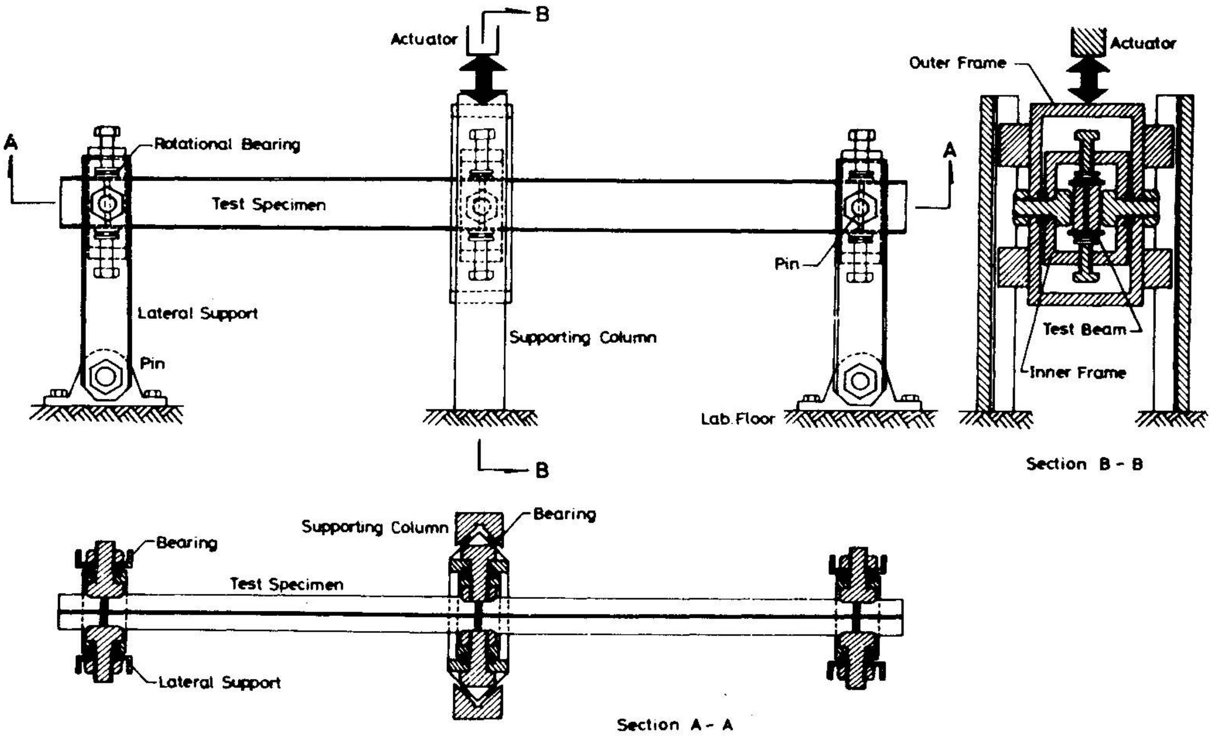


Fig. 1 General View of Test Setup

The specimens denoted by "monotonic" in Table 1 were loaded statically and monotonically. These static test results and the results previously obtained by the authors are compared to the results of reversed loading tests. The specimens denoted by "cyclic" in the Table were repeatedly loaded at constant deflection amplitudes and loading sequences were summarized in Table 1. The amplitudes were increased to the next step in each 10 cycles in general. The cycle number of 10 in each step was considered to be sufficient to exclude the transient property of material after sudden change of loading.

In order to examine the influence of the number of cycles in each amplitude on the critical amplitude, DG-130-7 and DG-130-8 were loaded each 5 and 15 cycle at each amplitude, respectively.

Table 1 Summary of Test Results

| Specimen | $l/r_y$ | Loading Condition | Frequency (Hz) | Cyclic Program Amplitude θ/θp (Number of Cycles) | Rotation Capacity ( R ) |
|---|---|---|---|---|---|
| DG-110-1 | 51.2 | Cyclic | 0.03 | 1.0—2.0—2.5—3.0—3.5<br>(10) (") (") (") (15) | 2.5 |
| DG-110-2 | 51.3 | " | " | 1.0—1.5—2.0—2.5—3.0—3.5—4.0—4.5<br>(10) (") (") (") (") (") (") (5) | 2.0 |
| DG-130-1 | 60.9 | Monotonic | —— | | 5.4 |
| DG-130-4 | 62.5 | Cyclic | 0.03 | 1.0—2.0—2.5—3.0—3.5<br>(10) (") (") (") (") | 2.0 |
| DG-130-5 | 62.5 | Earthquake Response[1] | 0.067[2] | 1.0—2.0—2.5—3.0—3.5—4.0—3.5—3.0[3]<br>(1) (") (") (") (2) (") (1) (1) | |
| DG-130-7 | 62.4 | Cyclic | 0.05 | 1.0—2.0—2.5—3.0—3.5—2.0—2.5—3.0—<br>(5) (") (") (") (") (") (") (")<br>3.5—2.0—2.5—3.0<br>(5) (") (") (") | 2.0 |
| DG-130-8 | 62.6 | " | " | 1.0—2.0—2.5—3.0—3.5—2.0—2.5—3.0—<br>(15) (") (") (") (5) (15) (") (")<br>3.5—2.0—2.5<br>(5) (15) (") | 2.0 |
| DG-130-9 | 62.6 | Random[4] | 0.06[2] | 1.0—2.0—2.5—3.0—3.5—4.0—4.5—2.0—<br>(1) (") (") (") (") (") (") (")<br>2.5—3.0—3.5—4.0—4.5—2.0—2.5—3.0—<br>(1) (") (") (") (") (") (") (")<br>3.5—4.0—4.5—2.5—3.0[3]<br>(1) (") (") (2) (") | |
| DG-130-10 | 62.6 | " | " | 1.0—2.0—2.5—3.0—3.5—3.83—4.5—2.0—<br>(1) (2) (") (") (") (") (") (")<br>2.5—3.0—3.5—3.83[3]<br>(2) (") (") (") | |
| DG-150-2 | 70.0 | Cyclic | 0.05 | 1.0—1.5—2.0—2.5—3.0—3.5<br>(10) (") (") (") (") (5) | 1.7 |
| DG-170-1 | 79.4 | Monotonic | —— | | 3.9 |
| DG-170-3 | 81.7 | Cyclic | 0.03 | 1.0—1.5—2.0—2.5—2.75—3.0<br>(10) (") (") (40) (10) (5) | 1.0 |
| DGH-110-1 | 52.2 | Monotonic | —— | | 3.5 |
| DGH-110-2 | 52.3 | Cyclic | 0.03 | 1.0—1.5—2.0<br>(10) (") (") | 1.3 |
| DGH-130-1 | 62.2 | Monotonic | —— | | 3.3 |
| DGH-130-2 | 61.9 | Cyclic | 0.05 | 1.0—1.5—2.0—2.5—3.0<br>(10) (") (") (") (6) | 1.1 |
| DGH-150-1 | 72.0 | Monotonic | —— | | 2.4 |
| DGH-150-2 | 71.6 | Cyclic | 0.05 | 1.0—1.5—2.0—2.5<br>(10) (") (") (") | 1.0 |
| DGH-170-1 | 80.7 | Monotonic | —— | | 1.5 |
| DGH-170-2 | 80.9 | Cyclic | 0.03 | 1.0—1.5—2.0—2.5<br>(10) (") (") (3) | 0.5 |

1) Response of deflection calculated with acceleration of ground motion recorded at Hachinohe 1968 earthquake

2) Mean frequency of response waves or random waves

3) Ratio of maximum rotation to θp

4) Random deflection generated on a digital computer

The deflection was controlled so as to be a sinusoidal function of time, t, in the "cyclic" tests. As the displacements of real structures do not show the simple harmonic response during earthquake, DG-130-5, DG-130-9 and DG-130-10 were loaded by controlling the central deflection to coincide with the calculated deflection response and the random deflection generated on a digital computer.

The frequency of cyclic deflection was determined to be 0.03 or 0.05 Hz due to the capacity of the power supply, but these low frequencies were convenient to measure force and deflection and moreover, the validity of predicting dynamic behaviors by low frequency tests was already established by Hanson(5,6) and Rea et al.(7).

## 3. EXPERIMENTAL RESULTS

The end rotation, $\theta$, and the end moment, M, defined by Fig. 2 were calculated from the central deflection and the applied force in each test. In order to compare the results in the same scale, $\theta$ and M are divided by $\theta p$ and Mp, respectively.

$\theta p$ is given by the expression

$$\theta p = MpL/3EI_x \qquad (1)$$

where Mp is full-plastic moment and $I_x$ is moment of inertia with respect to strong axis.

According to the traditional way to express the rotation capacity often appeared in the papers on the plastic design, the rotation capacity, R, was calculated by

$$R = (\theta/\theta p)_m - 1 \qquad (2)$$

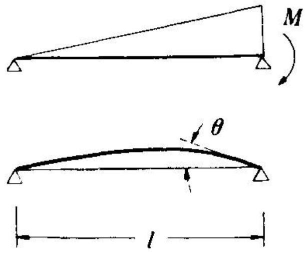

Fig.2 Definitions of Moment and Rotation

In the monotonic loading tests, $(\theta/\theta p)$ of the above expression was defined as the value of rotation, $\theta/\theta p$, at the maximum moment. The results are summarized in Table 1 and plotted in Fig. 3.

In the case of cyclic bending tests the moment-rotation hysteresis curves were obtained as shown in Fig. 4 where inserted numerals beside loops denote the numbers of cycles. As recognized by the figure, the hysteresis loops could keep their very stable shapes as long as the amplitude was controlled within some critical bound. Once the amplitude exceeded the critical bound, lateral buckling was accelerated, the hysteresis loops became unstable and the magnitude of M/Mp decreased in each cycle.

The failure due to lateral buckling is defined in this series of experiments as follows: After a few cycles at the same amplitude the moment reduces and moreover the rate of reduction does not vanish asymptotically and diverges.

The critical amplitude is defined as the amplitude, beyond which beam fails, and denoted by $(\theta/\theta p)_m$ to compare with results of the monotonic loading tests. R is defined by Eq.(2). The rotation capacity, R, obtained for specimens in the cyclic tests are summarized in Table 1 and plotted in Fig. 3. The solid curve in the figure is a regression curve which was made to be fitted to the data of the monotonic tests. It can be seen that R for the cyclic loads are considerably lower than for the monotonic loads.

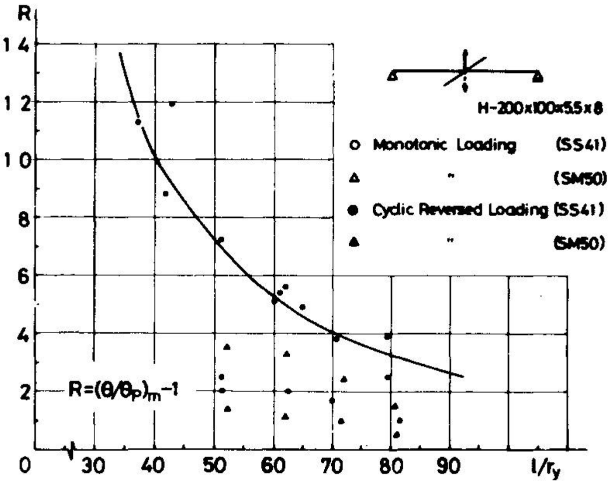

Fig. 3 Rotation Capacity-Slenderness Ratio

For specimens DG-130-7 and DG-130-8, the cyclic tests were done again at the smaller amplitudes than the critical amplitude, $3\theta/\theta p$, after the excessive buckling deformations took place at the larger amplitudes than the critical. In this loading program the stable hysteresis loops were obtained at the small amplitudes, but the reduction of the values of recorded bending moments was remarkable and it was influenced by the amount of residual buckling deformations.

To know what kinds of hysteresis loops and what behaviors of lateral

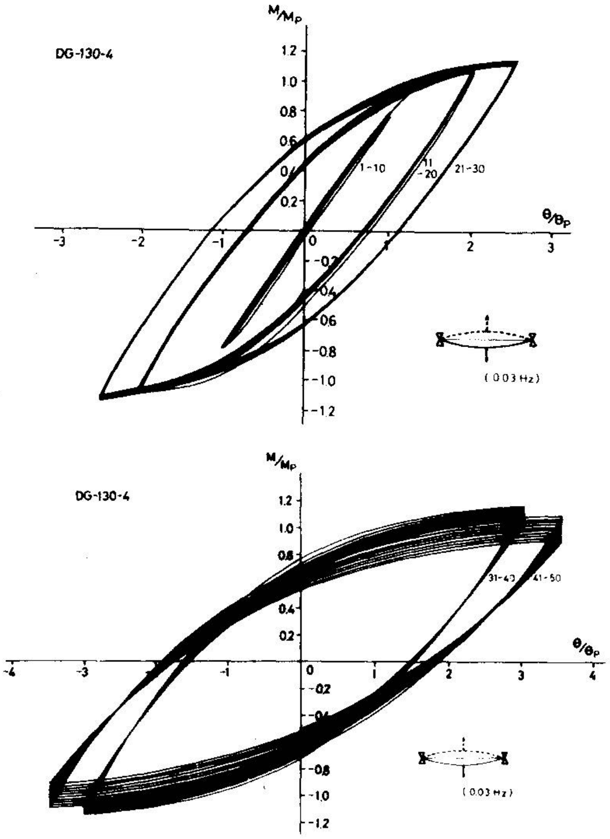


Fig. 4 Hysteresis Loops

buckling of real members show when structures are sujected to severe earthquakes, the reversed loading tests were done by controlling the deflection of DG-130-5, DG-130-9 and DG-130-10 to follow the calculated deflection response of a simple model and the random motion generated on the computer. Fig. 5 shows the recorded deflection of the center of span, $\delta$ , and the applied force, P, in DG-130-9, where the maximum value of the deflection was set to $3.5\theta/\theta p$. For comparison, the recorded deflection and force of DG-130-4 are also shown in the same figure. Figs. 6 and 7 show the hysteresis loops for DG-130-9. In Fig. 6 the maximum deflection was controlled to be equal to the critical amplitude, namely, $3\theta/\theta p$. In this case the load carrying capacity is considered to be enough because the amplitudes except the maximum are smaller than the $3\theta/\theta p$. And even though the maximum was set to $4\theta/\theta p$, the reduction of the moment at each peak is not so much. When the maximum became $4.5\theta/\theta p$, however, the remarkable reduction was observed at the maximum peak and the second maximum peaks which were also beyond the critical, $3\theta/\theta p$.

## 4. CONCLUSIONS

1) There exists the critical amplitude of deflection, within which the hysteresis loops are stable under cyclically repeated loads and beyond which the load carrying capacity of the beam reduces in each cycle. The same critical amplitudes are obtained for the different numbers of cycles in each amplitude if the geometric and material properties are same.
2) These critical amplitudes are considerably low in comparison with the deflections at the maximum loads in the monotonic loading tests. It is worth to be noted that the critical values for cyclically reversed loadings are lower

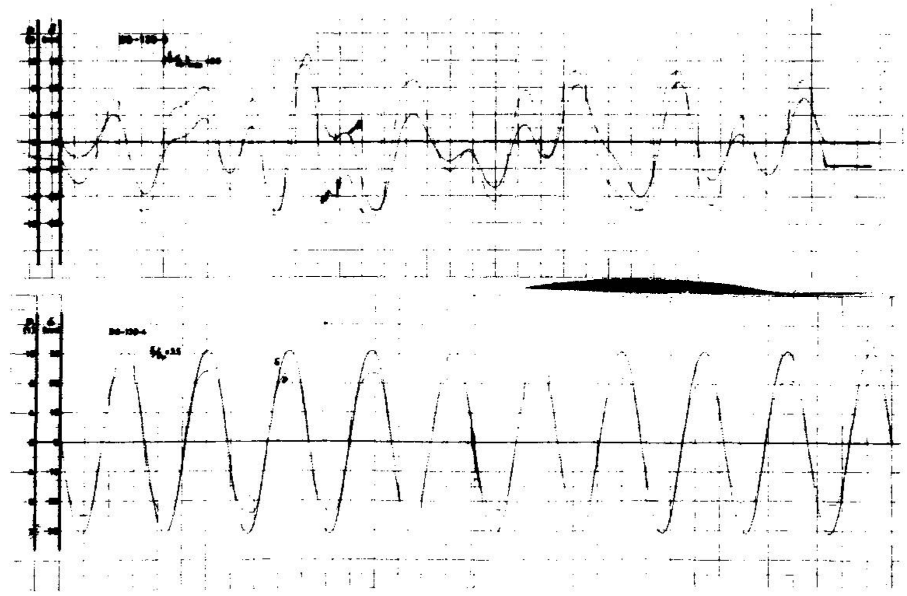

Fig. 5 Deflection and Load Recorded

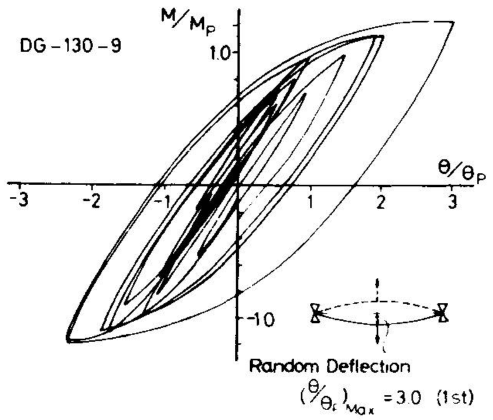


Fig. 6 Hysteresis Curves

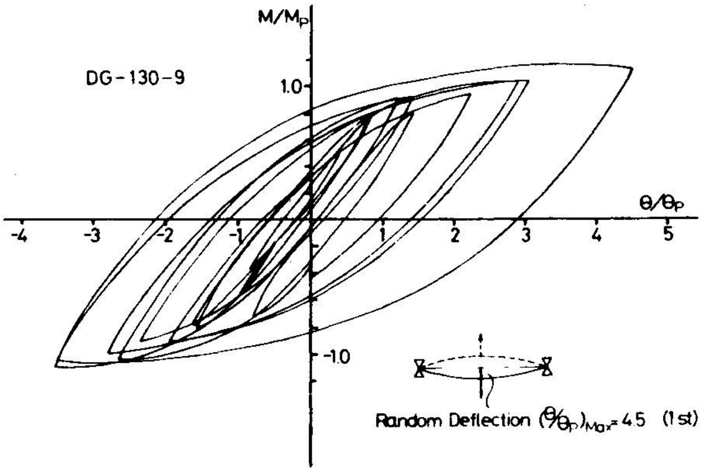


Fig. 7 Hysteresis Curves

than a half of the critical values for monotonic loadings.

3) Even though the beams were subjected to excessive deflections beyond the critical amplitude in the past, the stable hysteresis loops can be obtained if the deflection is within the critical . However, the load carrying capacity at that stages reduces depending on the amount of deflections over the critical amplitude and its number of experiences hitherto.
4) The critical amplitudes obtained by the cyclic tests are significant for analyzing the buckling failure due to random loads. For random loads only one excursion of hysteresis curve beyond the critical amplitude does not imply an immediate failure.
5) For SM50 steel, more severe bracing requirements than for SS41 steel for lateral buckling must be provided to beams of structures subjected to both monotonic and cyclically repeated loads.

## REFERENCES

1) V.V. Bertero and E.P. Popov, " Effect of Large Alternating Strains Steel Beams ", Proc. ASCE, ST 1, Feb. 1965
2) E.P. Popov and R.B. Pinkney, " Cyclic Yield Reversal in Steel Building Connections ", Proc. ASCE, ST 3, March 1969
3) " AIJ Specifications for Plastic Design of Steel Structures (Draft) ", Architectural Institute of Japan, 1971
4) " AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings ", Feb. 1969, American Institute of Steel Construction
5) R.D. Hanson, " Comparison of Static and Dynamic Hysteresis Curves ", Proc. ASCE, EM 5, Oct. 1966
6) R.D. Hanson, A.M. AlMuti and J.G. Hoad, " Post-Elastic Response of Mild Steel Structures ", Proc. 3rd. Japan Earthquake Eng. Symp. 1970
7) D. Rea, R.W. Clough, J.G. Bouwkamp and U. Vogel, " Damping Capacity of a Model Structure ", Proc. 4th World Conf. Earthq. 1969

## SUMMARY

Cyclically repeated loading tests at constant deflection amplitudes were conducted on the steel beams with H-shaped section. These tests showed the quite different behaviors of lateral buckling as compared with the behaviors under monotonically increasing loads. The rotation capacities for cyclic tests are considerably smaller than for monotonic tests. Reversed loading tests due to random deflections were also conducted.

RESUME

Des essais de charges répétées avec grandeurs de déformation constantes ont été effectués sur des poutres en acier à section en double T. Ces essais ont montré que le flambement latéral se produit d'une façon tout-à-fait différente du cas où la charge augmente lentement. Les possibilités de rotation pour les essais de fatigue sont beaucoup plus petites que pour les essais statiques. Des essais de charges alternées à la suite de déformations quelconques ont aussi été effectués.

ZUSAMMENFASSUNG

Zyklische Belastungsversuche mit konstant gehaltener Ausbiegung wurden an Stahlbalken mit H-Querschnitten durchgeführt. Diese Versuche zeigten das ziemlich verschiedenartige Verhalten des Kippens, verglichen mit dem Verhalten unter ständig zunehmender gleichgerichteter Belastung. Die Rotationskapazitäten für zyklische Versuche sind bedeutend kleiner als bei gleichgerichteten Versuchen. Versuche mit Wechselbelastung aufgrund von stochastischen Ausbiegungen wurden ebenfalls ausgeführt.

Leere Seite
Blank page
Page vide