

CUMULATIVE DAMAGE OF WELDED BEAM-TO-COLUMN CONNECTIONS
IN STEEL STRUCTURES SUBJECTED TO DESTRUCTIVE EARTHQUAKES

Kiyoshi KANETA (I)
Isao KOHZU (II)
Hidekazu NISHIZAWA (II)
Presenting Author: Isao KOHZU

SUMMARY

In order to assess aseismic safety of welded structures subjected to destructive earthquakes, based upon the concept of low cycle fatigue failure, this paper deals with the characteristics of low cycle fatigue of welded beam-to-column connections which were tested statically and dynamically under repeated loading conditions. From these experimental results, it has been recognized that low cycle fatigue damage of welded steel structures against destructive earthquakes can be well predicted by employing the proposed method derived from the statically loading test results.

INTRODUCTION

Moment resisting steel structures are well recognized as one of the most aseismic structures, because of good deformability beyond the elastic range and large amount of the energy absorbing capacity that the materials can possess. However, a large number of cyclic deformations in the inelastic range which the structures would suffer in the event of destructive earthquakes, may considerably result in a failure so-called the low cycle fatigue phenomenon at the critical sections in the structures.

From this point of view, it is necessary to make clear low cycle fatigue property of the materials, the subassemblages bolted or welded, and the whole structures, when the structures are subjected to such major earthquakes. Since the welded structures are commonly used in the current design practice, the inelastic behavior of the welded connections like as the beam-to-column connections, becomes the most important factor to assess aseismic safety of the structures. For this reason, this paper is focussed to the clarity of low cycle fatigue property and fatigue damage estimation of the welded beam-to-column connections due to earthquakes, based on the statical as well as the dynamical experiments.

STATIC LOADING TESTS

Specimens

Hot rolled sections H-200x100x5.5x8 and H-200x200x8x12 (unit; millimeter) made of the JIS SS41 grade steel were used for the beam and the column members, respectively. The coupon test results of the material used for the beam flange are shown in Table 1.

(I) Professor of Structural Engineering, Kyoto University, Kyoto, Japan

(II) Research Assistant, ditto

The welding of the beam-to-column connection was performed by means of the manual arc welding in the manner of which the web of the beam was fillet welded to the column flange and thereafter, each flange of the beam was butt welded to the column flange.

The specimens were classified into the following two series.

a) Series A

The specimens belonging to Series A were the cantilever type specimens of the same shape like those which had been tested by Popov et al.(Ref. 1), as shown in Fig.1. These specimens were further classified into three types, A1 to A3 types, according to the combinations of the programmed tip deflections and the welding procedures at the beam ends. For A1 and A2 type specimens, the beam flanges were single bevel butt welded with backing strips to the column flanges. For A3 type specimens, the beam flanges were butt welded with back run.

b) Series B

The specimens of Series B were fabricated as shown in Fig.2. The members of each specimen consisted of the two beams and the stub column connected to the beams at the center of the specimen. The panel of the column was stiffened by the doubler plates so as to prevent the shear deformations of the panel zone.

These details of the groove shapes are illustrated in Fig.3.

Procedures of the Experiments

The column of the cantilever type specimen was connected rigidly with high strength bolts to a reaction frame and the beam was loaded cyclically at the tip so as to generate the large bending moment at the beam end. The tip deflection of the beam was measured by an extensometer and the cyclic test was carried out in the manner to control the magnitude of the deflection as described later. The general view of the test set-up is illustrated in Fig.4. For the specimen of Series B, an universal loading machine was applied to load cyclically at the center of the specimen, by controlling the deflection at the center detected by an extensometer. The general view of the test set-up is shown in Fig.5. In addition, several wire strain gauges were pasted to the flange of the beam at or near the beam end, and they were continuously recorded on a digital strainmeter during the experiment.

Programmed deflection patterns consisted of two types, which were perfectly reversed types with and without mean deflections, and the combinations of the deflection patterns and the specimens were tabulated in Table 2 with the number of loading cycles repeated at the programmed deflection amplitudes or those to failure.

Test Results

The results of cyclic loading tests for the specimens indicated that the load-deflection curves and the local strain outputs stabilized at about 2 or 3 initial cycles and after many cyclic loadings the cracks initiated at the intersections of the beam flanges and the scallops, or at the welded connections of the beam flanges, and propagated along the directions of the flange

widths, as illustrated in Fig.6. It could be observed from the test results that the cyclic tests controlled by the deflection amplitudes at the tips or the centers of the specimens become equivalent to the strain controlled tests of the flanges at or near the beam ends and fracture patterns of every specimens do not depend upon the details of the connections and the magnitudes of the deflection amplitudes.

Predictions of the Fatigue Lives

a) Experimental Formula for Perfectly Reversed Loading Test Results without Mean Deflections

Since the mechanical properties of the materials are different from each other, their influence upon the fatigue life must be taken into consideration. For this reason, a method has been proposed to use the normalized plastic strain range instead of the directly measured plastic strain range, $\Delta\varepsilon_p$, in the following way: Assume that the initial stiffness remains unchanged until the point at which the beam end arrives at the fully plastic moment, M_p , then the corresponding plastic strain, ε_p , can be computed using the relation, $M_p/M_y = \varepsilon_p/\varepsilon_y$, in which M_y is the bending moment of the beam when the fibers of the flanges arrive at the yield strain, ε_y . Therefore, the non-dimensional plastic strain range, $\Delta\varepsilon_p/\varepsilon_p$, can be evaluated.

For the test results under the fully reversed constant deflection amplitudes without mean deflections, the relation between the non-dimensional plastic strain ranges and the corresponding fatigue lives can be formulated as follows by employing the least squares method.

$$\frac{\Delta\varepsilon_p}{\varepsilon_p} = 71 \cdot (N_f)^{-0.57} \quad (1)$$

In this equation, the plastic strain range, $\Delta\varepsilon_p$, was measured from the bending moment-averaged strain curve at the point of the beam from which the web was scalloped.

b) Fatigue Life Predictions for the Test Results under the Multi Step Deflection Amplitudes and the Amplitudes with Mean Deflections

A simplified prediction method referred to the hypotheses proposed by Miner, Weiss and other authors, was adopted to count the low cycle fatigue damage under varying strain amplitudes. Initially, it is assumed that cumulative damage can be estimated by linear summation of individual damage under constant strain amplitudes which can be defined as n_i/N_{fi} , in which n_i is an actual number of cycles under a constant strain amplitude, and N_{fi} is the number of cycles to failure under the corresponding constant strain amplitude. Since N_{fi} is calculated from Eq.(1), cumulative damage, DF , can be formulated as follows:

$$DF = \sum_i \frac{n_i}{N_{fi}} = \sum_i \frac{n_i}{\left(\frac{1}{K} \frac{\Delta\varepsilon_p}{\varepsilon_p} \right)^{0.57}}, \text{ in which } K = 71 \quad (2)$$

When the mean strain has to be taken into consideration, K value of Eq.(2) is substituted by K' as follows.

$$K' = K \cdot \left(1 - \frac{\epsilon_m}{\bar{\epsilon}} \right) \quad (3)$$

in which ϵ_m is the mean strain and $\bar{\epsilon}$ represents the value of $\Delta\epsilon_p$ which can be obtained by substituting N_f in Eq.(1) for $1/2$.

The computed results of the damage factors to failure for an individual specimen can be obtained as summarized in Table 3. From this table, it can be recognized that the estimate of fatigue lives under varying strain amplitudes can be assessed within the safety region by assuming that fatigue failure happens when the value of damage factor arrives at 0.5.

DYNAMIC LOADING TESTS

Specimens and Test Structures

In order to simulate the dynamic behavior in an actual weak beam-strong column type steel structure and the low cycle fatigue failure at the beam-to-column connection in the event of destructive earthquakes, a simple structure model was fabricated and tested on a shaking table by applying two types of excitations to the structure-shaking table system.

Geometry of the specimen used for this experiment is shown in Fig.7. The material was made of JIS SS41 grade steel. The mechanical properties of the plates used for the beam were obtained from the coupon test and the results are summarized in Table 4. The beam flange was single bevel butt welded with backing strip and the web was fillet welded to the column flange. The panel zone of the beam-to-column connection was stiffened by the doubler plates to prevent the shear deformation, and their detail is also shown in Fig.7.

A same column member as that of the specimen was used to form a complete frame structure together with the specimen and was pin-jointed to the free end of the beam. Two identical plane frames were spaced about 1.5 m apart and were erected so as to form a three dimensional one bay-one story steel structure on the shaking table. The column bases of the structure were pin-jointed to the steel footings fastened to the shaking table. The structure was reinforced at the both side elevations by angle bracings, and concrete blocks weighing about 1.35 tons connected rigidly to the roof beams were supported by the removable transverse beams so that the beams of the specimens could not be subjected to the vertical loads carried from the concrete blocks.

The beam of the specimen was supported by the cross beams at the third points to prevent lateral buckling. In addition, the roof was stiffened by diagonal braces of the tension rods in order to prevent torsional oscillations of the structure.

Procedures of the Experiments

The shaking table driven by an electro-magnetic type shaker, which was of capacity as listed in Table 5, was used to generate the horizontal excitations

Table 1 COUPON TEST RESULTS OF SERIES A AND B

	SPECIMEN TYPE		
	A ₁	A ₂ , B	A ₃
YIELD STRESS σ_y (τ/cm^2)	2.78	2.88	3.33
TENSILE STRENGTH σ_u (τ/cm^2)	4.10	4.63	4.65
YIELD STRAIN ϵ_y (%)	0.130	0.135	0.162
ELONGATION ϵ_u (%)	27.8	30.4	29.8
YOUNG'S MODULUS E (τ/cm^2)	2138	2133	2056

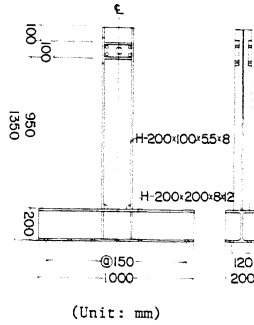


Fig.1 SPECIMEN (SERIES A)

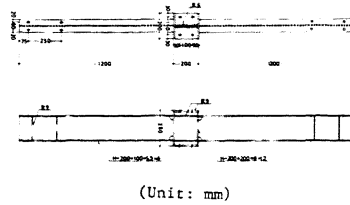
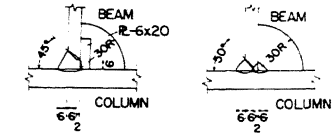


Fig.2 SPECIMEN (SERIES B)



TYPE A₁, A₂ AND B TYPE A₃
Fig.3 DETAILS OF GROOVE SHAPES

Table 2 PROGRAMMED DEFLECTIONS AND NUMBER OF CYCLES

SPECIMEN	PROGRAMMED DEFLECTION AMPLITUDE (cm)
A ₁ - 1	+1.26(53) [0.0]
A ₁ - 2	+1.00(189) [0.0]
A ₁ - 3	+0.74(605) [0.0]
A ₂ - 1	+0.74(20) + +1.00(10) - +1.26(10) +1.52(10) + +1.78(6) [0.0]
A ₂ - 2	+0.74(20) [0.37] - +1.00(10) [0.5] +1.26(10) [0.63] - +1.52(10) [0.76] +1.78(5) [0.89]
A ₂ - 3	+1.78(10) - +1.52(10) - +1.26(10) +1.00(7) [0.0]
A ₂ - 4	+0.74(3) - +1.00(3) - +1.26(3) - +1.52(3) +1.78(3) + +1.52(3) - +1.26(3) - +1.00(3) +0.74(3) - +1.00(3) - +1.26(3) - +1.52(3) +1.78(3) + +1.52(3) - +1.26(3) - +1.00(3) +0.74(3) - +1.00(3) - +1.26(3) - +1.52(3) +1.78(3)
A ₂ - 5	+1.00(10) [0.0] - +1.00(10) [0.5] +1.00(10) [1.0] - +1.00(10) [0.5] +1.00(10) [0.0] - +1.00(10) [-0.5] +1.00(10) [-1.0] - +1.00(10) [-0.5] +1.00(10) [0.0] - +1.00(10) [0.5] +1.00(10) [1.0] - +1.00(10) [0.5] +1.00(10) [0.0] - +1.00(10) [-0.5] +1.00(10) [-1.0]
A ₃ - 1	+1.26(127) [0.0]
A ₃ - 2	+1.52(51) [0.0]
A ₃ - 3	+1.00(211) [0.0]
A ₃ - 4	+1.26(94) [0.0]
A ₃ - 5	+1.78(39) [0.0]
B - 1	+1.68(32) [0.0]
B - 2	+1.00(135) [0.0]
B - 3	+0.60(20) + +1.00(10) - +1.68(10) +0.60(20) + +1.00(10) - +1.68(10) [0.0]
B - 4	+1.68(10) - +1.00(10) - +0.60(20) +1.68(7) [0.0]
B - 5	+2.30(4) - +1.68(8) - +2.30(4) [0.0]

(): NUMBER OF CYCLES, [] : MEAN DEFLECTION

Table 3 COMPUTED DFS OF SERIES A AND B

SPECIMEN	DF
A - 1	0.82
A - 2	0.67
A - 3	0.84
A - 4	0.85
A - 5	0.67
B - 3	0.86
B - 4	0.59
B - 5	0.72

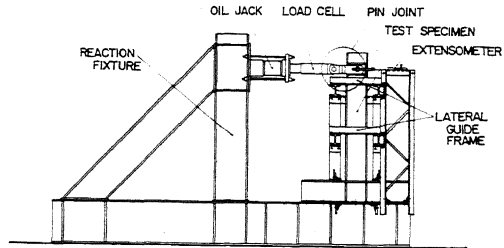


Fig.4 SET-UP OF LOADING APPARATUS FOR SERIES A

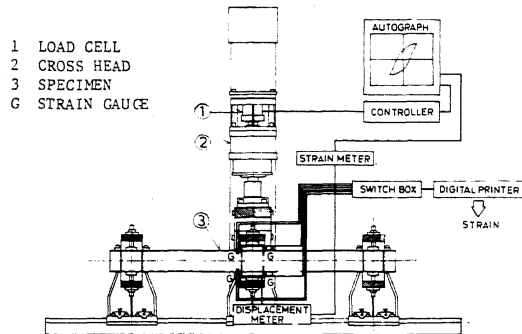


Fig.5 SET-UP OF LOADING APPARATUS FOR SERIES B

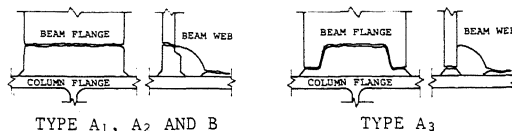


Fig.6 FRACTURE PATTERNS

