Proceedings
10th European Conference on Earthquake Engineering

28 AUGUST – 2 SEPTEMBER 1994/VIENNA/AUSTRIA

Editor
GERALD DUMA
Central Institute for Meteorology and Geodynamics, Vienna, Austria

OFFPRINT

A.A.BALKEMA/ROTTERDAM/BROOKFIELD/1995

10th European Conference on Earthquake Engineering, Duma (ed.) © 1995 Balkema, Rotterdam. ISBN 90 5410 528 3

Shaking table tests of composite steel-concrete, beam-column connections

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ABSTRACT: A series of nine real time earthquake-type dynamic tests have been performed, using an earthquake simulator, in order to study the effect of the beam to the column connection, to estimate the influence of the thickness of steel web in the shear panel to the behaviour of the composite steel-connections and the effect of the input motion.

1 INTRODUCTION

The work reported in this paper is part of a research programme on the earthquake response of beam-column connections of composite steel-concrete moment resisting frames. This programme is carried out by a team of research institutions from Belgium, Germany and Greece.

One of the aims of this programme is to investigate the reliability of static and pseudo-dynamic tests in capturing the earthquake response of composite structures. This aim will be achieved by comparing the results of the above mentioned tests with the results of tests performed on a shaking table.

In this paper the experimental results of tests that have been carried out on the shaking table at the Laboratory for Earthquake Engineering of National Technical University of Athens are reported.

2 THE SPECIMEN

The specimen, is a composite steel-concrete made of a column of a height of 3m and a beam of a length of 1.5m connected to the mid-height of the column. Six welded and three bolted specimens, as shown in Table 1, have been tested in order to study the effects of the type of the beam to the column connection, the thickness of the steel web in the shear panel as well as the input motion .

3 THE TESTING SET-UP

The experimental studies have been performed using the multy-hinged loading frame shown in Figure 1, that was specially fabricated in order to generate inertia forces. The loading frame is a mechanism and its stability is achieved, only, with the incorporation into it of the specimen forming a three hinged structure. The test system (loading frame plus specimen) is considered as possessing a one degree of freedom, with a concentrated mass at the top. At that level, the inertia forces generated by the mass are acting horizontally at the top hinge of the column.

The horizontal beam of the test system, is formed out of two pieces, rigidly connected one to the other. The one is a steel beam of known characteristics, W and E, while the other one is the horizontal beam of the specimen out of the composite material of unknown, yet, characteristics, section modulus and modulus of elasticity.

Table 1. Type of connection and thickness of web

	Thickness of steel web in the shear panel		
Connection	Normal t=11 mm	Reduced t=7 mm	Increased t=15 mm
Welded	W1-1 W1-2	W2-1 W2-2	W3-1 W3-2
Bolted	B 1-1	B 2-1	B 3-1

4 IDENTIFICATION OF DYNAMIC CHARACTERISTICS

The natural period and the damping of the specimen are identified by analysing the recorded signals in the frequency domain, as well as in the time domain. In the frequency domain the specimen is excited by a broad band random acceleration signal of a rather small amplitude. The acceleration

Figure 1. Experimental set-up

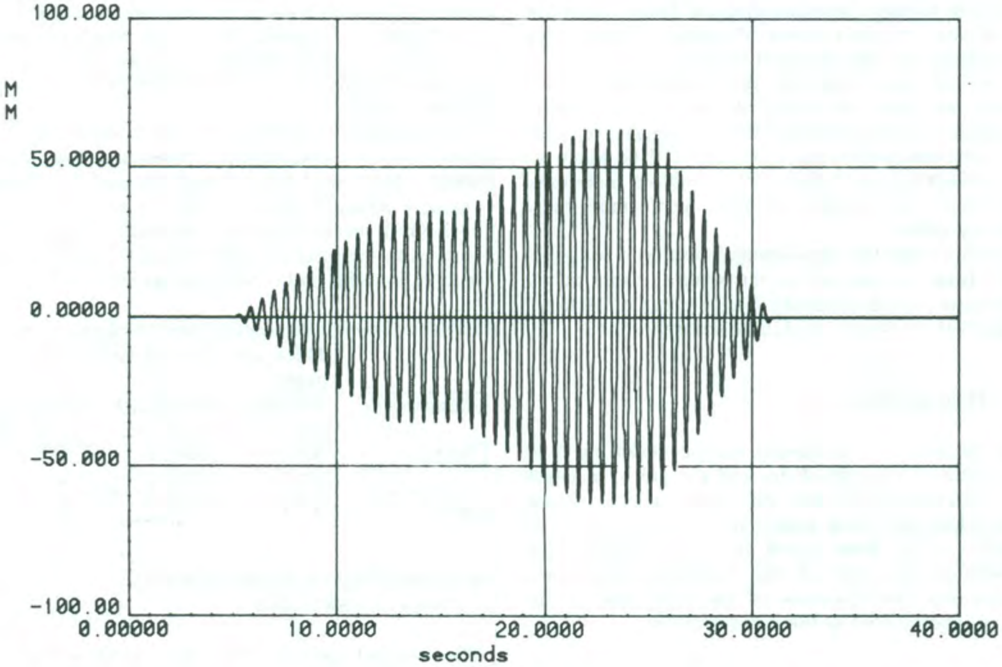

Figure 2. The time history of the desired excitation input motion at the base

at the level of the masses at the top is recorded and the transfer function is computed. The half power bandwidth method is used to evaluate the damping ratio. In the time domain the natural period is determined from the free vibration of the top of the specimen and the logarithmic decrement method is used in order to identify the damping ratio. The obtained results are shown in Table 2 for each specimen.

Table 2. Initial Period and Damping of the specimens

SPECIMEN	PERIOD (sec) (Trf)	PERIOD (sec) (Resp)	DAMPING RATIO (Trf)	DAMPING RATIO (Resp)
W1-1	0.460	0.465	0.0500	0.0400
W2-1	0.445	0.445	0.0500	0.0820
W3-1	0.445	0.465	0.0445	0.0385
B1-1	0.445	0.465	0.0445	0.0630
B2-1	0.465	0.493	-	-
B3-1	0.465	0.462	-	0.0460
W1-2	0.440	0.446	0.0500	0.0800
W2-2	0.445	0.440	0.0450	0.0500
W3-2	0.450	0.446	0.0445	0.0370

5 THE INPUT MOTION

Two types of ground motions were used in the shaking table tests: a sinusoidal type excitation with a linearly increased amplitude and an earthquake type excitation. The time history of the former ground displacement (see Figure 2) is analytically described as follows:

$a = 0.60$ cm

$T = T_0/0.80$

T_0 = Natural period of the test specimen

Zone I (ascending envelope, number of cycles=13)

$$u(t) = at\mathrm{Sin}(\frac{2\pi t}{T}) \quad (1)$$

Zone II (constant envelope, number of cycles=6):

$$u(t) = u_{\max(I)}\mathrm{Sin}(\frac{2\pi t}{T}) \quad (2)$$

Zone III (ascending envelope, number of cycles=10):

$$u(t) = u_{\max(II)}\mathrm{Sin}(\frac{2\pi t}{T}) + at\mathrm{Sin}(\frac{2\pi t}{T}) \quad (3)$$

Zone IV (constant envelope, number of cycles=6):

$$u(t) = u_{\max(III)}\mathrm{Sin}(\frac{2\pi t}{T}) \quad (4)$$

Zone V (descending envelope, number of cycles=10):

$$u(t) = (u_{\max(IV)} - 2at)\mathrm{Sin}(\frac{2\pi t}{T}) \quad (5)$$

The total duration of the excitation is defined at each test according to the natural period of the respective test specimen (dependent on the number of cycles mentioned above).

For the selection of the earthquake type input motion used in the two last tests W2-2 and W3-2, the procedure was as follows:

After the selection of a set of real earthquake ground motions, the records were modified through a single degree of freedom system with a natural period of 0.46 sec and a damping coefficient ζ=2%, before being used as input motion for the final tests. The natural period and damping of the SDOF system were selected in order to correspond to a natural period accounting for an initial stiffness degradation of the specimen used, in order to have a boost of the response of the specimen after its initial yielding, and the estimated damping coefficient of the specimen.

For the final selection of the input motion a further analysis was performed. The modified records were used as input motions to a single degree of freedom system, having a natural period of 0.445 sec, damping coefficient ζ=2% and yield acceleration (force/mass ratio) 1.5g. The inelastic response followed a stiffness degrading model with a hardening ratio of 5% as it was calculated from the tests results with the sinusoidal excitation. The earthquake that required the maximum ductility was selected and scaled so that the shaking table was functioning at its capacity.

6 THE INSTRUMENTATION

The data acquisition system Quad Conditioner of MTS with 64 channels was used. A low pass filter of 30 Hz was used for all recordings. The sampling rate was 100 pps. In Figures 3 and 4 The general instrumentation is presented.

7 CALCULATION OF THE INTERACTION DIAGRAMS M/θ_{test}, M/θ_s, M/θ_{tot}

The various member forces are calculated from data measured on the steel beam. For the calculation of the moment of the horizontal steel beam at a length l from the hinge, the following formula has been used:

$$M_l = \frac{E(\varepsilon_1 - \varepsilon_2)}{2}W \quad (6)$$

where: $\varepsilon_1, \varepsilon_2$ are the strains of the top and bottom flange of the beam, respectively

W is the section modulus

E is the modulus of elasticity of the material of the beam

for our case, for steel, E=205.000 N/mm^2 and W=836 cm^3 which leads to:

$$M_l = (\varepsilon_1 - \varepsilon_2) \times 8.569 \times 10^{10} (\mathrm{N\,mm}) \quad (7)$$

The vertical reaction force at the hinge is:

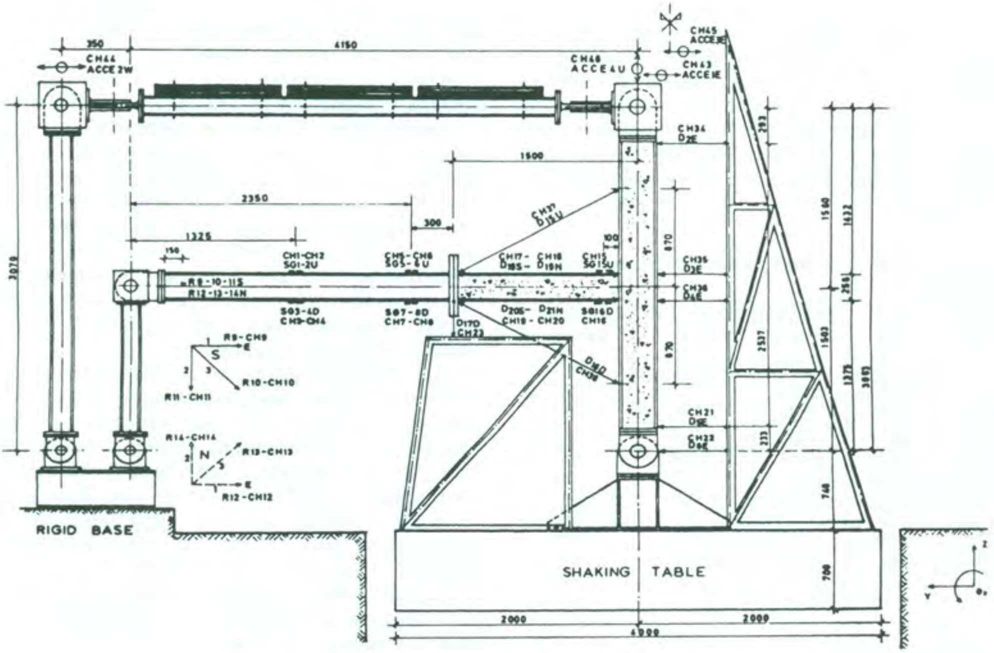

Figure 3. Test apparatus and instrumentation set-up

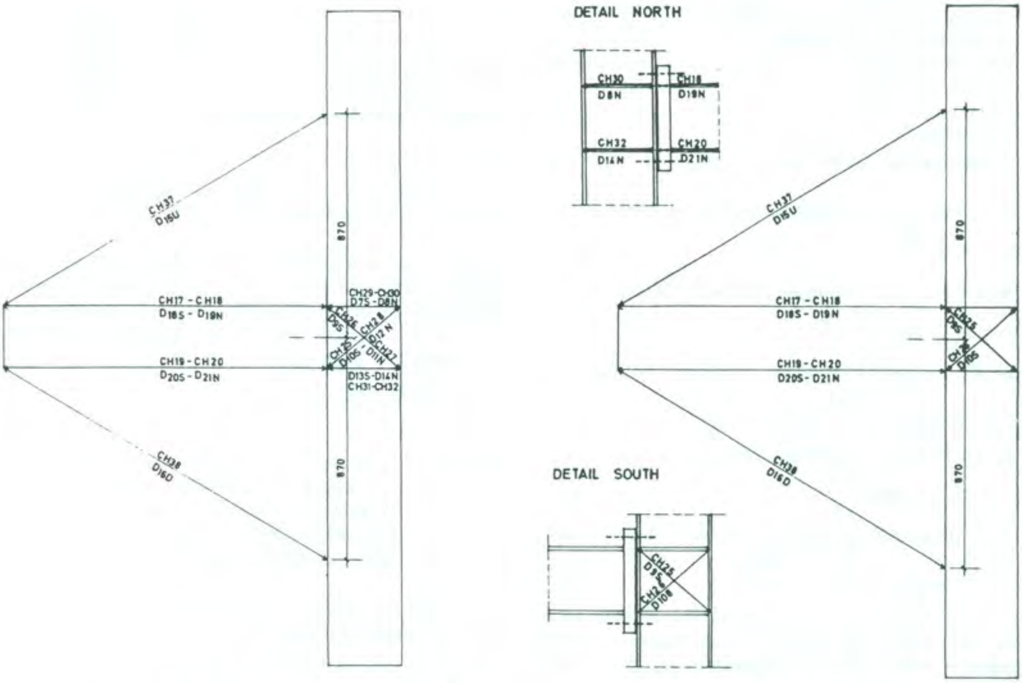

Figure 4. Detailed instrumentation set-up (a) welded specimen (b) bolted specimen

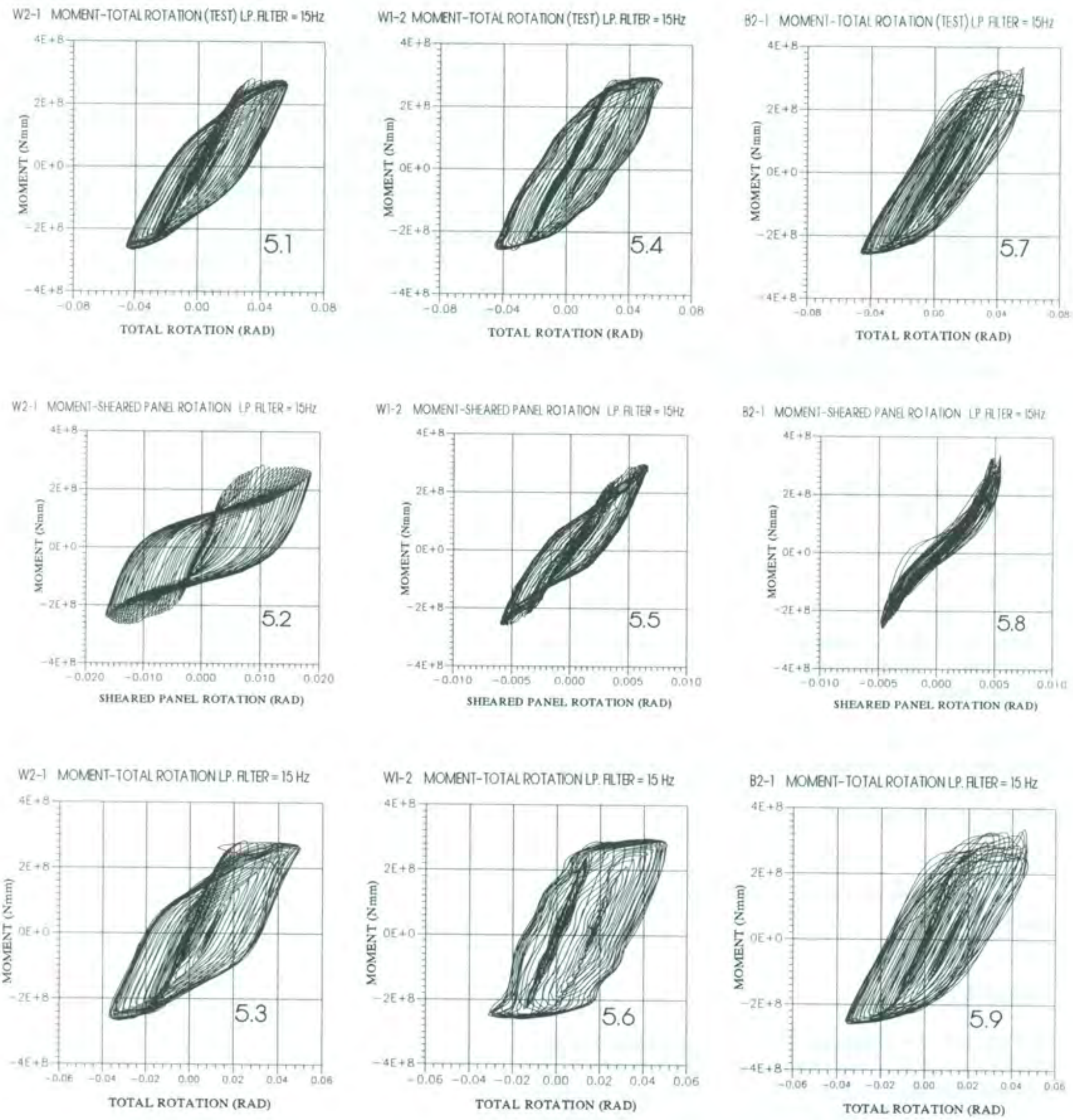

Figure 5. M/θ_{tot},M/θ_{test} and M/θ_s interaction diagrams for W2-1 ,W1-2 and B2-1 tests

$$V = \frac{M_1}{l} = \frac{(\varepsilon_1 - \varepsilon_2)}{l} \times 8.560 \times 10^{10}\,(N) \qquad (8)$$

where l is the distance of the measured strains ε_1 and ε_2 from the hinge of the beam in (mm).

The test piloting rotation θ_{test} is calculated after the formula:

$$\theta_{test} = \frac{D2E - D5E}{l} = (D2E - D5E) \times 3.94 \times 10^{-4} (9)$$

The sheared panel deformation θ_S is calculated, after the formulae:

$$\theta_S = (D20S - D9S)\ 0.0026 \qquad (10)$$

or

$$\theta_S = (D12N - D11N)\ 0.0026 \qquad (11)$$

The total rotation θ_{tot} is calculated , after the formula:

$$\theta_{tot} = (D15U - D16D) \times 6.8 \times 10^{-4} \qquad (12)$$

Table 3. Characteristic values of the test results

SPECIMEN	My (kNm)	θy (rad)
W1-1	240	0.013
W2-1	230	0.017
W3-1	256	0.015
B1-1	240	0.016
B2-1	224	0.012
B3-1	250	0.014
W1-2	245	0.014
W2-2	235	0.016
W3-2	251	0.015

8 ESTIMATION OF THE ACTIVE MOVING MASS AT THE TOP OF THE SPECIMEN

Using the simple expressions of

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{m}{K}} \Rightarrow m = \frac{T^2}{4\pi^2}K \qquad (13)$$

where

$$K = \frac{H}{\delta} \qquad (14)$$

where H is the horizontal force at the top of the specimen, corresponding to the horizontal displacement δ.

The relation between the vertical reaction V of the horizontal beam, measured according to the formula (8), at the left hinge and the horizontal force H at the top of the specimen:

$$H = \frac{V}{3063}4150 = 1.3549V \qquad (15)$$

The weight of the mass at the top is found to be equal to 5.6 ton.

9 RESULTS

In Figure 5 the diagrams of bending moment versus total rotation θ_{tot}, and the test piloting rotation θ_{test} and sheared panel rotation θ_s are presented, for the tests W2-1, W1-2 and B2-1. In Table 3 the elastic limit of M_y and the corresponding total rotation are also presented. M_y corresponds to the intersection between the elastic slope and the line tangent to the plastic branch, having a slope of 1/10 of the elastic one.

10 CONCLUSIONS

In case of damages in the beam, the column or the panel, the damaged steel flanges were buckled in an antimetrical way (i.e., for the beam, the buckled flanges were at the top left hand side and at the bottom right hand side, or vice versa).

As the panel was formed stronger, the damages were moved towards the beam and the column. The opposite was also observed: the weaker (thinner steel web panel thickness) the panel, the more damages were concentrated to it, releasing thus beam and column.

As a consequence of this observation, in the case of weaker flange, the total ultimate deformation of the system is mainly due to the deformation (rotation) of the sheared panel.

Comparing the bolted to the welded specimens, the whole deformation was due to flexure deformation of the beam, while the sheared panel rotation was negligible.