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Results and correlative analysis of a composite two storey eccentric braced frame test

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Abstract: This paper covers an experimental study of the behaviour of a large-scale, two-story eccentrically braced composite frame (EBF) with steel composite beam and column sections and composite floors connected to the beams by means of headed shear - studs. The overall length of the two bay test frame (with bracing in only one bay), was 10.0 m (32.8 ft) and the overall height was 5.50 m (18.0 ft). The experimental response covering the frame behaviour under alternating cyclic horizontal loads involving both increasing and subsequently decreasing displacement load histories is presented. Correlative analytical studies to capture the derived experimental behaviour are discussed.

1 INTRODUCTION

Extensive research on steel seismic-resistant Eccentrically Braced Frames (EBFs) began in 1975. Roeder and Popov (1977) proved, that in comparison with conventional framing systems, EBFs (see figure 1a) meet two basic criteria of seismic design: high stiffness under service loads and large ductility under overload levels. The typical energy dissipation mechanism for EBFs under severe earthquake loading is shown in figure 1b. In such EBFs the vertikal components of the brace member forces are held in equilibrium by the shear and bending moments in short beams of length e - the shear links. These links deform inelastically when the structure is overloaded, thereby dissipating considerable energy. EBFs have become a widely accepted structural system used to resist seismic forces.

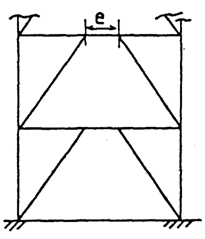

Fig. 1a K - braced EBF

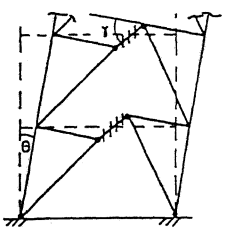

Fig. 1b Deformed EBF

In Western Europe steel-concrete composite construction, including steel rolled sections with infilled concrete between the flanges in either side of the web, has been studied extensively, and has found increasingly broad application, particularly also because of the inherent fire resistance in comparison with bare steel structures.

Considering the potential advantages of composite eccentrically braced lateral load resistant systems in particular, an extensive experimental programm on steel-concrete composite behaviour under earthquake simulated conditions has been carried out by Bouwkamp at the Technical University of Darmstadt, Institute of Steel Construction since 1989. One of the main aspects of the EBF system studies was to assess the efficiency of infilled concrete rather than typical web stiffeners in preventing web buckling of the shear links.

Recognizing that the effective behaviour of the link under earthquake loads in general is of utmost importance, and that the behaviour of steel EBFs with composite floors has been investigated by Ricles and Popov (1987), the major research effort at Darmstadt has been directed to the behaviour of the composite link region.

One research effort covered a study of a two bay, two story composite EBF. In the following, information about the test specimen, test setup and test performance, as well as experimental results and a correlative analysis are presented.

2 TEST SPECIMEN

The two-bay, two-story, composite eccentrically braced frame with a one-bay bracing arrangement had overall dimensions of 2 x 5.00 m (2 x 16.4 ft) in length and 2 x 2.75 m (2 x 9 ft) in height (Fig.2). Based on earlier composite beam column tests, steel-cross sections of HE 260 A for the beams and HE 300 B for the columns were also choosen in this case. A concrete slab with a width of 1.00 m (3.3 ft) and a thickness of 0.12 m (4.75 in) was connected to the girder by means of welded shear studs, resulting in a composite floor-beam section (Fig. 3). The slab was reinforced with two Q 221 (2.21 cm^2/m) bi-axial reinforcing mats, positioned near the top and the bottom of the slab. To meet the fire-resistant requirements 4 ø 20 longitudinal rebars and ø 8 stirrups at 15 cm intervalls were placed between the flanges of both beam and column sections (Fig. 3). At the base of the columns heavy plates were provided and connected to the heavy base beam by HS-bolts. This connection was designed elastically. To ensure a secure and ductile behaviour of the column base regions, the infilled concrete was omitted from the lowest portion of the column over height of 30 cm.

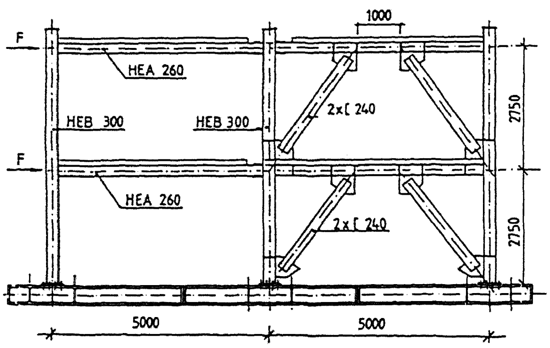

Fig. 2 Testspecimen

In order to study the resistant capacity of the composite shear link within the available horizontal test-load capacity, the overall lateral load resistance of the test frame was intentionally lowered by reducing the column-base moment resistance. This was achieved by reducing the width of the column flanges over a certain distance immediately above the base plates.

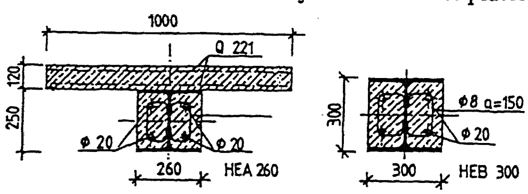

Fig. 3 Cross Sections

The ductility of eccentrically-braced frames in general is concentrated in the shear links located in the mid-portion of the beams between adjacent braces. Depending on the lenght of the link, either shear- or endmoment yielding will deliver the required ductility. In this case, with the link designed to exhibit shear yielding, not only the steel beam-web but also the composite infilled concrete would contribute basically to the shear stiffness and overall resistance. Instead of the normal stirrup spacing of 15 cm in the link region the stirrups had 5 cm intervalls in order to confine the infilled concrete very well (Fig. 4). Because of uncertainty about the degree of participation of the filled-in concrete in the overall shear-link resistance, the box-section braces were intentially overdesigned and were formed by 2 x U 240 channel sections, interconnected by two side-plates 12 x 200 (0.5 in x 7.8 in), fillet welded over the full length of the member. In order to ensure an elastic behaviour of the brace-connection during the ultimate response of the system, the braces were welded to edge- stiffened concentric gusset plates. The test-load transfer into the braced bay was ensured by connecting the beam flanges through full penetration welds to the interior columns. However, the connections between beams and exterior columns were designed as HS-bolted shear tabs.

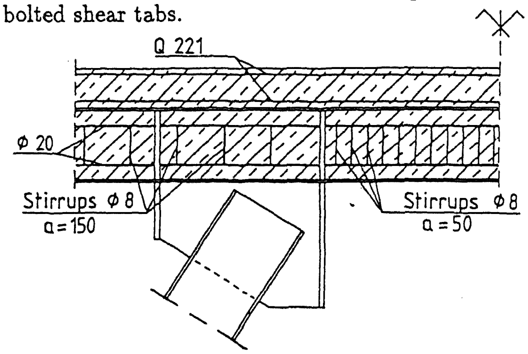

Fig. 4 Link Detail

3 TEST-SETUP

For the test performance a special reaction frame had been designed. The column base plates of the test frame were HS- bolted to a heavy base-beam (HD 400 x 400 x 551). This beam in turn was pre-stressed to the laboratory tie-down slab and was connected by means of transverse girders to the two vertical, parallel, reaction trusses. These trusses were interconnected by means of transverse girders located at the load introduction levels (floor levels). The horizontal test loads were introduced at each floor level by means of double acting actuators with maximum force capacities of about 960 kN in tension and 1300 kN in compression. The upper and the lower actuators had double-amplitude capacities of 130 cm and 100 cm, respectively.

The lateral position of the test setup had been ensured by a stability frame which provided guidance to the test frame at four points at each floor level. Both the lower beam flanges as well as the edges of the concrete slab were lateraly held in position at locations adjacent to the exterior columns.

4 TEST PERFORMANCE

4.1 General

In order to determine the cyclic force-displacement characteristics of the EBF assembly in both the elastic and non-linear ranges, the test specimen was subjected to displacement-controlled forces. In principle, the upper floor level was displacement controlled, while the associated horizontal force at that level was used to force-control the actuator at the lower floor level at 50 % of the recorded upper floor force. Reflecting the considerable stiffness of the excentrically braced frame the elastic alternating cyclic top-floor displacements to be introduced initially were set at maximum values of 2, 4, 6 and 8 mm, respectively. Subsequently, it was intended to introduce horizontal displacements with increasing steps of 8 mm, thus resulting in total alternating displacements of 16 mm, 24 mm, 32 mm, 40 mm, etc. at the top-floor level. These displacements were to be applied three times at each displacement increment. Unfortunately, the above intended testing schedule could not be executed because of load capacity limitations in tension (pulling of cylinders). Instead an alternative test procedure was selected during the actual testing.

Accordingly, the testing procedure was divided into two phases. During the first phase of the test, the load at the lower floor level was load-controlled as intended (50% of upper floor load). However, this load condition limited to study the EBF fully in the inelastic range. Hence, after top-floor displacements of 32 mm had been reached, it was decided to alter the load arrangement by setting the lower actuator load at 100 % of the top floor load (phase 2). In this load setting, the second and third test sequence was performed until a significant loss of stiffness of the frame had been observed.

4.2 First Test Sequence (Phase 1)

The load-displacement diagram during the first phase is shown in figure 5. In this phase it was not possible, because of load limitations, to introduce the intended displacement time-history exposure. As a result, at first under tension only a maximum displacement of 16 mm was found, later on because of general loss of resistance a maximum displacement of + 20 mm was reached; under compression (pushing the frame) a horizontal displacement of - 32 mm could be fully reached. After this displacement level had been reached three times, it was decided to reduce the displacement in single cycles of +/- 24, 16 and 8 mm, respectively. This displacement load-history reflected a decreasing earthquake exposure, and was deemed important to assess the cyclic response of the EBF before further introducing larger displacements into the inelastic load range. The results of the single-cycle tests with step-wise smaller displacements, clearly indicate the loss of stiffness (resistance) of the composite shear link following a previously higher displacement exposure. The same single cyclic-alternating loading was followed also in order to bring the test frame back to the previously attained maximum displacement of - 32 mm. Subsequently the original testing procedure was continued with introducing three alternating displacement cycles of - 40, - 48 and - 56 mm, respectively. In the third cycle at - 56 mm the load resistance had only dropped to about 1.80 MN, showing a quite stable hysteretic behaviour of the frame. Following this load sequence the test was stopped at a zero level displacement.

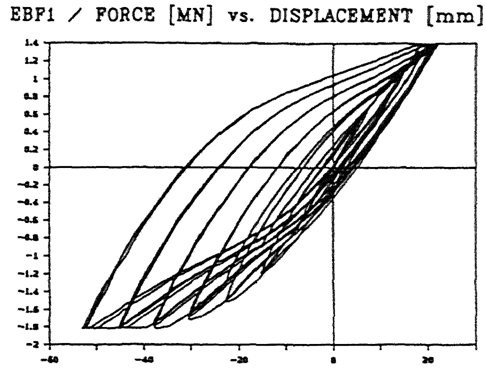

Fig. 5 Load-Displacement Diagram

4.3 Second and Third Test Sequence (Phase 2)

With the previously noted increased actuator capacity of about 1:1 between lower and upper actuators, it was intended to introduce a full cycle of - 56 to + 56 mm. Although it was not possible to reach the + 56 mm displacement, the frame was subsequently subjected to a hysteretic cyclic exposure with step-wise reduced displacements, reflecting the tail-end of an earthquake.

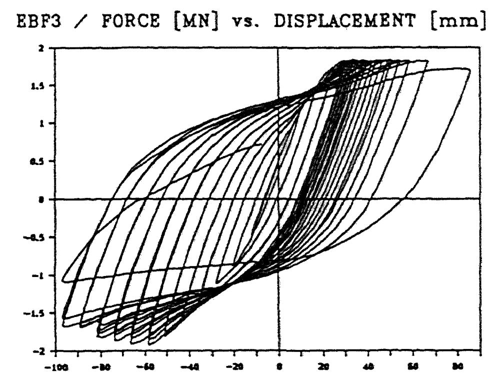

Fig. 6 Load-Displacement Diagram

With a final offset of - 6 mm (at zero load) the results clearly showed the efficiency of the brace frame under reduced earthquake effects following a substantial overstressing of the system. In the final test sequence (initial displacement offset of about - 6 mm) the EBF was first subjected to single hysteretic load cycles of 8 and 16 mm (Fig. 6). Subsequently, the test frame was subjected incrementally (8 mm intervalls) to alternating increasing nominal displacements from 24 mm up to 96 mm (each applied 3 times). Hereby, the prescribed displacements in the negative zone (pushing; total capacity 2.6 MN) could be achieved without difficulty. In tension the forcing was again limited by the load capacity (+ 1.9 MN). Hence, displacements increased only marginally from 24 to 56 mm. Only after substantial failure of the shear link had occured, the tensile load capacity was sufficiently large to introduce full cyclic displacements of nominally 64 mm and larger.

5 OBSERVED FAILURE MECHANISM

The behaviour of the EBF shows, that the main energy dissipation was concentrated in the beam shear-links. Despite the intentionally reduced column cross sections at the base plates, only yielding in the link regions could be observed throughout most of the test. This development signified a shear yielding of the steel beam web and was associated with diagonal cracking of the beam filled-in concrete (resulting in a concrete truss action). The concrete slab cracked very early during the test. A significant loss of stiffness and resistance which was observed near the end of the test could, after post-test removal of the concrete in the link zone, be attributed to a shear tearing of the beam web near the bottom flange adjacent to the vertical stiffener. Only in the final phase limited yielding occured in the reduced lower column sections as well as at the semi-rigid shear tab connections in the unbraced bay.

6 MECHANICAL FORMULATION

6.1 Material Properties

The material properties were determined by tension coupon tests of the steel-section material and concrete compression tests. The values are shown in the following table.

	Yield Strength kN/cm^2	
	Girder	Column
Conc. inf.	2.90	2.90
Conc. slab	2.18	-
Flanges	27.7	26.6
Web	34.9	32.0

Tab.1: Evaluated Material Properties

6.2 Elasto-Plastic Capacity

The plastic resistant capacity of the frame under increasing horizontal loads will be mainly determined by the shear resistance of the link zones. During the test an elasto-plastic behaviour of the shear-link zone was observed. To capture this behaviour in the structural model the following contributions have been examined, namely: the bare steel link section, the infilled concrete and the shear resistance of the concrete slab.

6.2.1 Bare Steel Link Behaviour

Tests of bare shear links, like the work of Popov, Kasai (1987) showed for properly stiffened links an excellent ductile stable shear link behaviour with a ductility ratio as high as $\mu \approx 30 - 40$. In that case the shear yield force is determined as:

$$V_p = 0.55\sigma_y \cdot d \cdot t_w \tag{1}$$

and the shear distorsion γ as:

$$\gamma = \frac{\sigma_y}{\sqrt{3} \cdot G} \tag{2}$$

At the same time considerable isotropic strain hardening, as high as 50% has been observed for shear links subjected to very large cyclic inelastic rotations ($V_u/V_p \approx 1.5$).

The cyclic behaviour of steel section materials under normal stress has been investigated by Ramberg/Osgood. They gave the cyclic stress strain relationship for different types of steel under cyclic loading with

$$\epsilon = \frac{\sigma_y}{E} + \left(\frac{\sigma_y}{K}\right)^{\frac{1}{n'}} \tag{3}$$

with the form factors K and n' depending on the yield stress of the steel. Seeger (1989) carried out several tests to determine the form faktors K and n' for different materials which had been used in this correlative analysis. Considering that,

$$\gamma = \frac{\tau}{G} = \frac{\sigma \cdot 2 \cdot (1+\mu)}{E \cdot \sqrt{3}} \tag{4}$$

$$\sigma = \frac{V \cdot \sqrt{3}}{A_s} \tag{5}$$

the authors derived the following formula describing the relationship between shear force V and shear distorsion γ under cyclic loading

$$\gamma = 1.5011 \left(\frac{V \cdot \sqrt{3}}{A_s \cdot E} + \left(\frac{V \cdot \sqrt{3}}{A_s \cdot K}\right)^{\frac{1}{n'}}\right) \tag{6}$$

Meanwhile the normal stress strain hardening effects gives a factor of appproximately 2 times of the yield stress σ_y for σ_u, while the accompanied shear hardening effects leads only to 1.5 times of τ_y for τ_u. Therefore the formula can be modified to

$$\gamma = 1.5011 \left(\frac{V \cdot \sqrt{3}}{A_s \cdot E} + \left(\frac{2 \cdot V \cdot \sqrt{3}}{1.5 \cdot A_s \cdot K} \right)^{\frac{1}{n'}} \right) \qquad (7)$$

Considering the specific material properties a $V - \gamma$ relationship for the steel panel as shown by curve (a) in figure 8 has been proposed.

6.2.2 Infilled Concrete

To describe the behaviour of the infilled concrete in the shear link a truss action as shown in figure 7 has been considered. In using the relationship between the truss action and the associated shear distorsion, the link behaviour could be analysed using the nonlinear material model for reinforced concrete as described by Park and Kent (1971). With a maximum compressive strength f_c at a strain ϵ of 0.002, the shear distorsion accordingly is $\gamma = 0.004$. Typically, the severe decrease of strength after reaching this strain level has been accounted for as shown in curve (b) in figure 8. The maximum shear force $V_{concrete}$ can be calculated using the following equations

$$A_{strut} = (b_{concrete} - t_{web}) \cdot d_{strut} \qquad (8)$$

$$V_{y,concrete} = D_y \cdot \cos\alpha = f_c \cdot A_{strut} \cdot 0.5 \cdot \sqrt{2} \qquad (9)$$

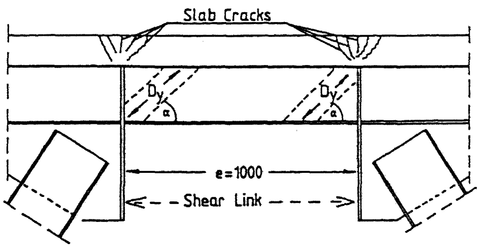

Fig. 7 Concrete Strut

For the subsequent correlative analysis the concrete strut resistance for the first part of the test has been calculated with the full width of the infilled concrete. However, in the correlative analysis for the the third sequence only the confined concrete between the stirrups has been considered (because of concrete spalling). Considering the specific properties and sectional dimensions figure 8 shows the $V - \gamma$ relationship for the first test sequence.

6.2.3 Concrete slab

Tests involving vertical shear in continuous composite beams, performed by Johnson (1971), indicate that if longitudinal reinforcement, provided in the slab, has not yielded, this reinforcement will carry a portion of the vertical shear. Once the longitudinal reinforcement yields, the steel rolled section is forced to provide the total vertical shear resistance. Observations during the test showed slab cracking due to bending very early. Therefore in this analysis the influence of the slab is neglected.

6.3 Elasto-Plastic Behaviour

The superposition of the above noted different contributing effects describing the elasto-plastic link behaviour under shear distortion is shown in figure 8. Whereby curve (a) represents the load-shear distortion for a link including the isotropic strength hardening of the steel web and curve (b) the same for the concrete behaviour, curve (c) represents the sum of these effects. The actual overall behaviour of the shear link as used for the calculation is represented by the bi-linear curve (d).

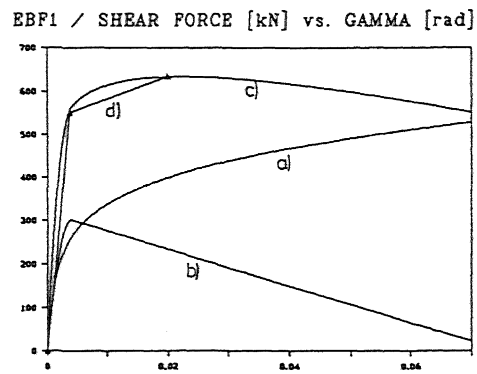

Fig. 8 Elasto Plastic Link Behaviour

7 CORRELATIVE ANALYSIS

The columns and girders were modeled as FEM beam-column elements. The elastic footing point behaviour has been represented by an additional element with a suitable bending stiffness (reflecting the column fixity). The reduced yield moment in the column was assumed to occur at a point $0.5 \cdot d_c$ above the base plate. This assumption reflects the structural detail of the decreased column-flange width introduced to reduce the overall stiffness of the test frame. The geometry of the braced-end connections was captured in the computer model by introducing end eccentricities. The influence of the brace member connections was captured by introducing for the braces an effective, reduced cross-sectional area. The elastoplastic behaviour of the link as shown in curve (d) in figure 8 was captured by a normal beam-column element with special yield conditions. Using this structural model and the nonlinear structural analysis program ANSR, the cyclic deformation response has been analized. Results are shown in figures 9 and 10 and showed an excellent agreement between experiment and analysis.

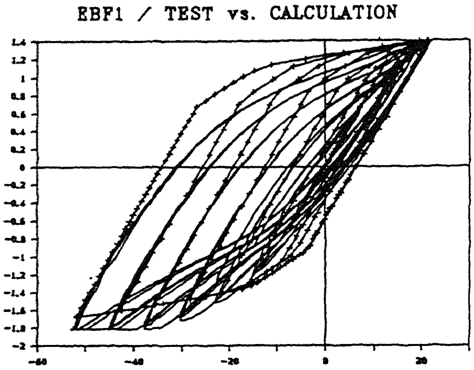

Fig. 9 Analytical Response 1. Test Sequence

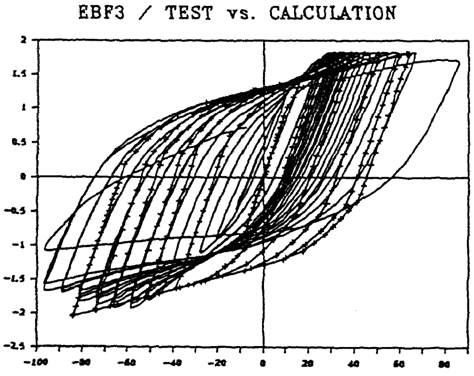

Fig. 10 Analytical Response 3.Test Sequence

8 CONCLUSIONS

Results of a composite eccentrically braced frame subjected to alterningly increasing displacements showed an excellent ductile response, concentrated in the shear link. Also, tests of the frame under cyclically reduced displacements - reflecting the behaviour after extreme earthquake exposure - exhibited a response whereby the structure, despite extreme local plastification in the shear link region, did return to its original position. The composite behaviour of the concrete in the shear links, proved to be most beneficial and showed that a closely spaced stirrup arrangement is an effective replace of the steel stiffeners typically used in bare steel EBFs.

A correlative analysis were carried out for the two different phases of the test. The results showed, that the behaviour of the composite shear link was captured very well under using a concrete strut model to capture the influence of the infilled concrete while neglecting the influence of the concrete slab for shear resistance.

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