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Seismic response of a cast-in-place steel fibre concrete joint connecting precast beams and columns

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ABSTRACT: An experimental investigation to evaluate the response of a monolithic precast joint, with cast-in-place steel fibre reinforced concrete (SFRC), when subjected to earthquake-type lateral loading is described in this paper. The use of SFRC in monolithic precast joints subjected to high magnitudes of static and dynamic actions is desirable due to its isotropic nature. Design variables that control the response of a SFRC joint subjected to lateral loading were evaluated, with special attention given to the influence of these parameters on the cast-in-place joint between the double-tee beams and the column. Response of the test specimen was also examined in terms of beam plastic hinge spreading.

It was found that this joint, when using SFRC, possessed greater strength and deformation capacity than when constructed of ordinary concrete, despite incorporating a smaller quantity of conventional reinforcement. The joint developed only superficial damage at later stages of testing, corresponding to the development of full beam plastic hinging.

1 INTRODUCTION

Experience from earthquakes, and extensive laboratory testing, have shown that well designed, detailed and constructed cast-in-place reinforced concrete frames perform satisfactorily during severe seismic events. In order to resist earthquake effects, the design of building structures including precast concrete elements, such as shown in Figure 1, must provide a practical method for connecting the precast elements together such that seismic performance will be as for a monolithic structure.

Steel fibre reinforced concrete is isotropic. Consequently, it appears conspicuously rational to use cast-in-place steel fibre concrete for structural elements that are subjected to high magnitudes of static and dynamic actions (ACI 1988, 1993 and RILEM 1977). The research reported here contributed to a better understanding of the seismic response of precast beam-column connection behaviour and the advantages of SFRC utilisation in monolithic joints (Tuleasca 2000). This knowledge will allow designers in seismic regions to safely take advantage of opportunities involving the use of SFRC in potential plastic zones of structural elements.

2 DESIGN OF TEST SET-UP

By testing a joint that used SFRC to connect precast beams to precast columns, it was intended to obtain information regarding joint behaviour, when longitudinally directed seismic actions were applied to a structure (Fig. 1). Cuciureanu and Mihul (1987) conducted testing considering the performance of precast double-tee beams and precast columns connected using a peripheral joint comprised of non-fibrous concrete. The test rig used for the research reported herein was similar to that previously used by Cuciureanu and Mihul. To demonstrate the superior behaviour characteristics of SFRC (Filiatrault et al. 1995), an interior joint was tested at a 1:1 scale. The experimental aim was to obtain a pseudo-static loading scheme that modelled real structure behaviour.

Figure 1 Multi-storey building structure

The adopted solution (see Fig. 2) used longitudinal beams (LB), with double-tee cross section and a flange height equal to the height of concrete floor, with the two web cores of the longitudinal beams "wrapping" the column (C) and being supported by the transverse beams (TB). The longitudinal double-tee beams were manufactured with projecting hairpin reinforcement (2). The necessity of satisfactory interaction between the precast double-tee beams and the column, permitting transmission of important bending moments, led to the use of a SFRC circumferential joint about the column perimeter, ensuring rigid response of the connection between the structural elements. The column was made in one piece (450×500 mm), with corbels for transverse beams seating, and projecting reinforcement (1) for connection to the precast transverse beams.

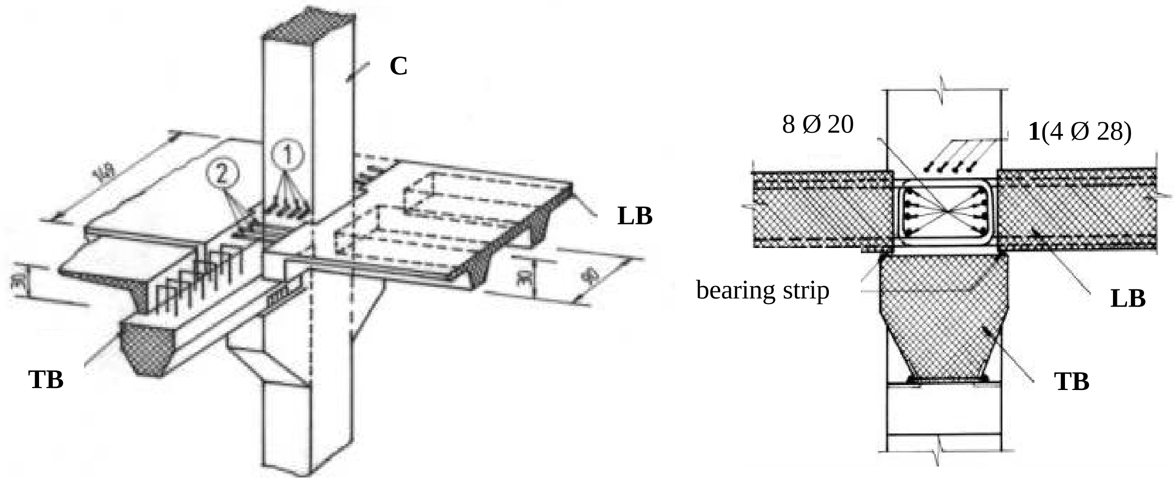

Figure 2 Schematic of joint assemblage

Construction details are shown in Figure 3 for the column, in Figure 4 for the transverse beam, and Figure 5 for the longitudinal double-tee beam. All precast elements used concrete designated C 20/25 (Eurocode Standard) indicating a characteristic cylinder compression strength of $f_c^{'} = 20$ MPa or a characteristic cube compression strength of $f_c^{'} = 25$ MPa. Three kinds of reinforcement were used: PC 60 (high carbon steel, 600 MPa rupture strength) with $f_y = 350$ MPa as longitudinal reinforcement, STNB (undeformed steel mesh) with $f_y = 370$ MPa as reinforcement for the double-tee beam slab (M1, M2 in Fig. 4) and OB 37 (low carbon steel, 370 MPa rupture strength) with $f_y = 210$ MPa for the stirrups. A concrete topping was not applied to the longitudinal beams in order to more clearly identify the specific performance of the SFRC circumferential joint.

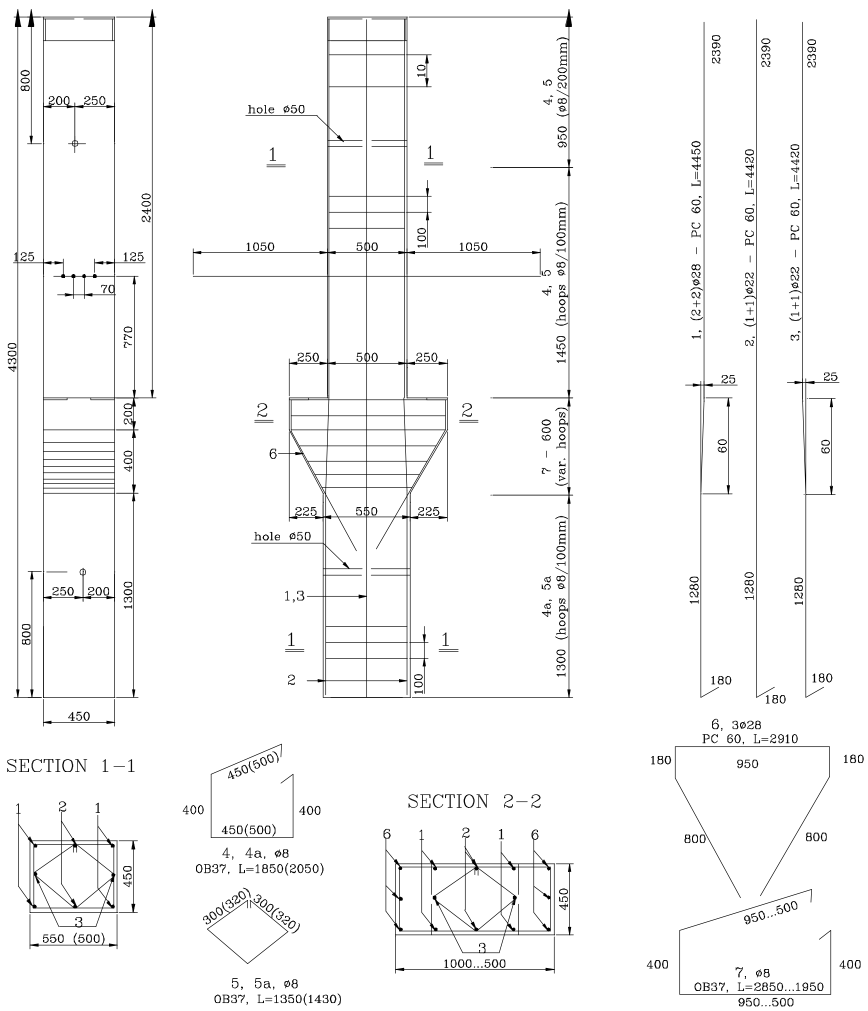

Figure 3 Column construction details

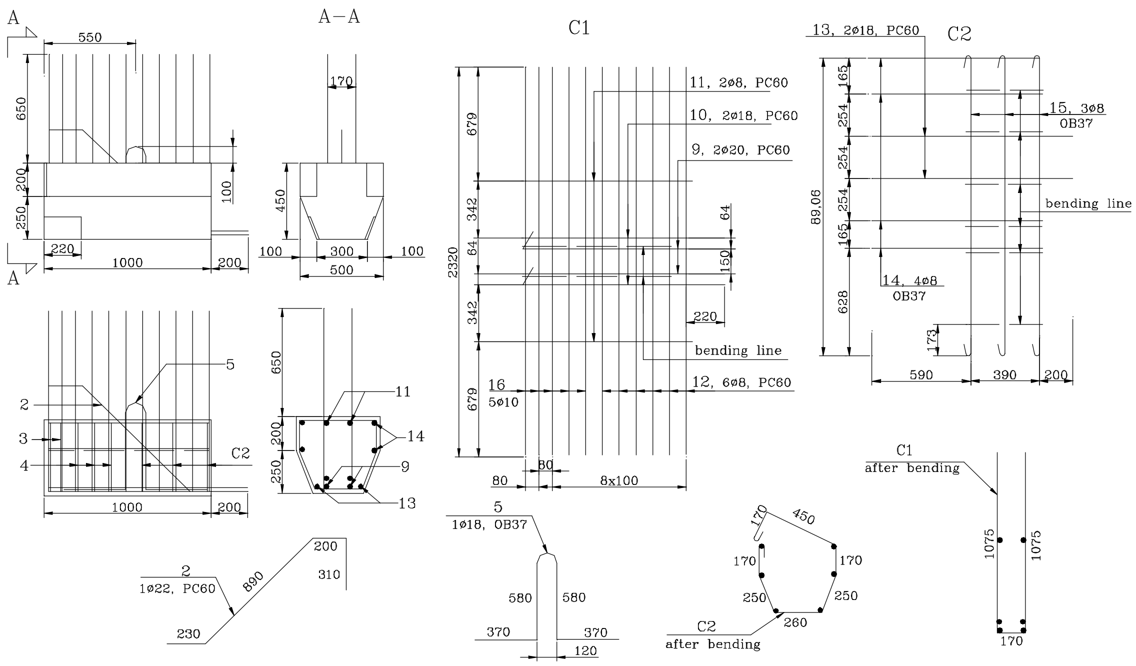

Figure 4 Transverse beam construction details

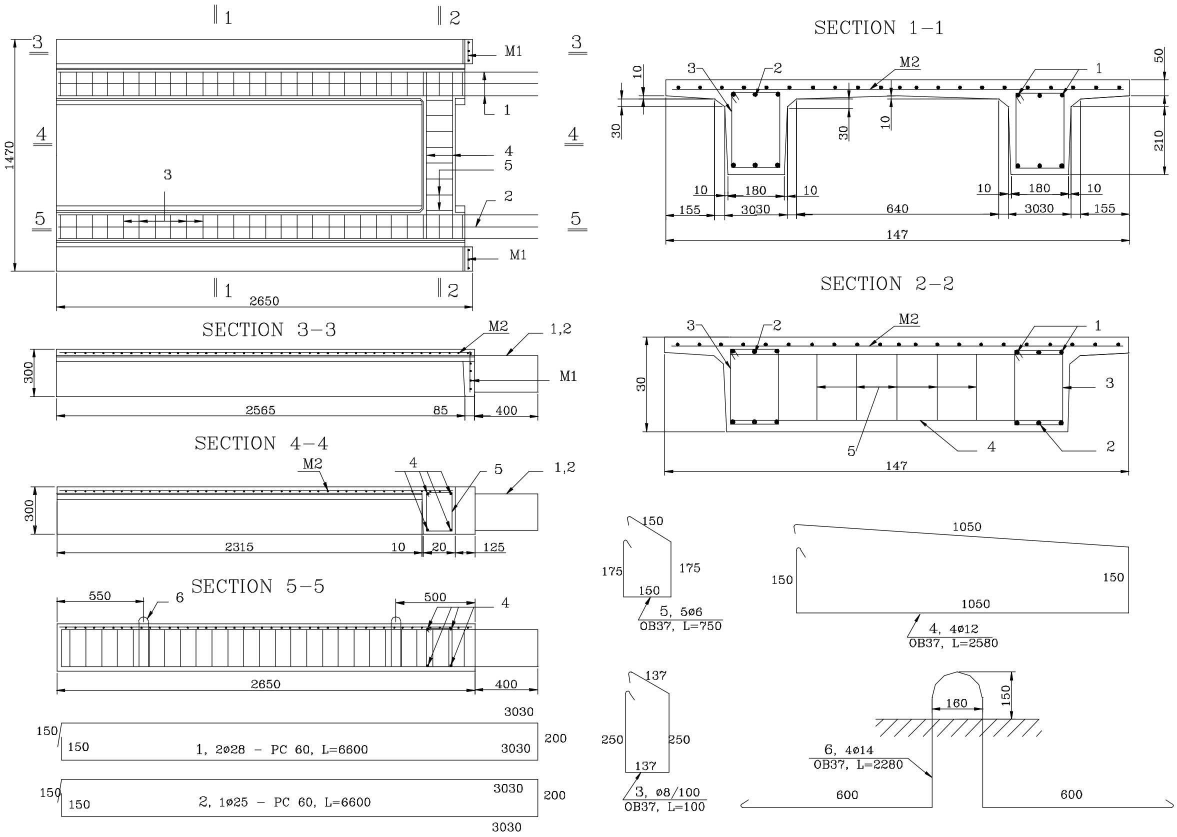

Figure 5 Longitudinal double-tee beam construction details

The test rig used in this study is shown in Figure 6 and consisted of:

? a steel foundation;

? two test frames supported by tilted tie bars and provided with transverse beams of 2.00 m span at the +5.00 m height, where a longitudinal 7.00 m length beam was fixed with steel yokes;

? horizontal crossed tie bars placed at an elevation of +4.00 m.

Figure 6 Test rig blueprint

Loads were applied at the ends of the two longitudinal beams and on top of the column, as shown in Figure 7.

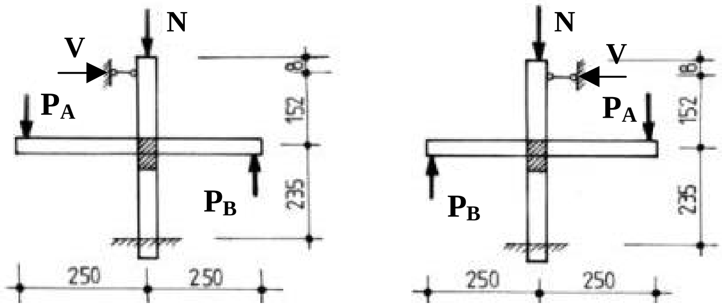

Figure 7 Adopted cyclic loading scheme for test

Using the dimensions and reinforcement details given in Figures 3, 4 and 5 and the characteristic material strengths referred to earlier it was found for the longitudinal double-tee beams that $M_n^- = 227.7$ kNm, $M_n^+ = 257.5$ kNm, $M_y^- = 189.4$ kNm, and $M_y^+ = 199.4$ kNm. This corresponded to $P_n^- = 99.8$ kN, $P_n^+ = 113.2$ kN, $P_y^- = 83.3$ kN, and $P_y^+ = 87.7$ kN.

3 EXPERIMENTAL PROGRAM

Designers of the structure shown in Figure 1 provided the envelope of critical actions at the interface between the longitudinal beams and the column of the interior joint.

The experimental program was divided into four stages as shown in Figure 8. The first three stages considering three, four and two cycles of loading to the Serviceability Limit State response of an interior joint located at different heights in the building:

? cyclic loading ($P_{A,\,max} = P_{B,\,max} = 45$ kN, N=380 kN) for an interior joint at the 3rd floor;

? cyclic loading (three cycles to P = 65 kN and one cycle to $P_{max} = 75$ kN, N = 600 kN) for an interior joint at the 2nd floor;

? cyclic loading (one cycle to P = 90 kN and one cycle to $P_{max} = 100$ kN, N = 800 kN) for an interior joint at the 1st floor;

The last stage of the experiment considered a semi-cycle of loading to the damage control limit state loads ($P_{max} = 120$ kN, N = 800 kN) projected to develop for an interior joint located at the first floor height.

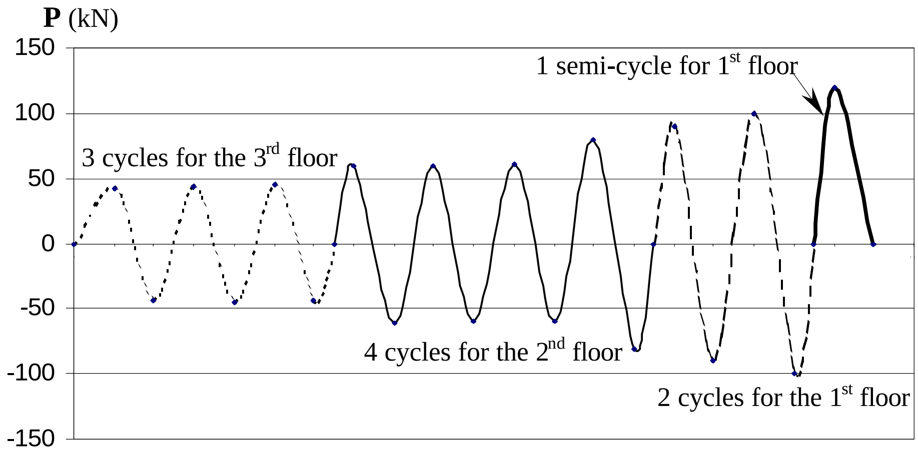

Figure 8 Loading history

Corrections were applied to the maximum/minimum moments considered as test starting points, in order to take into account the bending moments produced by the self-weight of the cantilever. For each of the three initial stages, axial forces applied to the column corresponded to the average axial compression values for the level above and below the considered joint.

Cantilever displacements were measured using a theodolite. Supplementary measurements of the cantilever displacement were made using displacement inductive sensors. These were placed two by two at the end of each cantilever. A further eight inductive sensors were used to measure opening of the interface between the joint and the longitudinal beams.

4 EXPERIMENTAL RESULTS

4.1 Experimental joint behaviour at the Serviceability Limit State

The position of instrumentation readings is shown in Figure 9. In each case, measurements taken from each of the webs of a longitudinal beam were averaged, and biased against the measurement obtained at the joint (point 5), which was measured to have less than 1 mm of deformation throughout the test.

The joint rotation was measured using the average of two reference points positioned above the transverse beams (reference marks 10-11, 12-13 from Fig. 9).

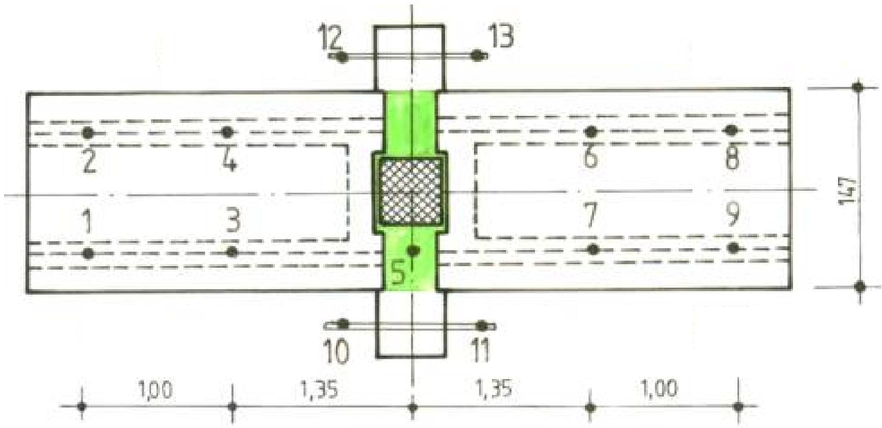

Figure 9 Reference points

Joint behaviour under alternating seismic type actions emphasised the following aspects:

- ? First stage - cracks appeared in the lower part of the longitudinal beam with a maximum width of 0.05 mm, which closed upon unloading. The residual deformations were small (~ 5 mm).
- ? Second stage – cracks appeared in the upper part of the longitudinal beam with maximum widths of 0.15 mm in the 4th cycle, 0.2 mm in the 5th cycle and 0.23 mm in the 6th cycle. These cracks closed upon unloading. The residual deformations increased to approximately 10 mm.
- ? Third stage – cracks in the cast-in-place steel fibre concrete developed on the top surface, reaching a maximum of 0.27 mm in the 7th cycle, 0.31 mm in the 8th and 0.36 mm in the 9th cycle. These crack widths compare favourably with the results previously reported by Cuciureanu and Mihul (1987), who measured maximum crack widths in their non-fibrous concrete joint of 5 mm. Upon unloading the cracks remained partially open. The residual deformations increased to approximately 21 mm.

At this stage, cracks previously formed in the transversal rib of the longitudinal beam opened to about 0.15 mm, indicated elasto-plastic behaviour of the longitudinal beams. Figure 10 shows the condition of the test unit at the Serviceability Limit State.

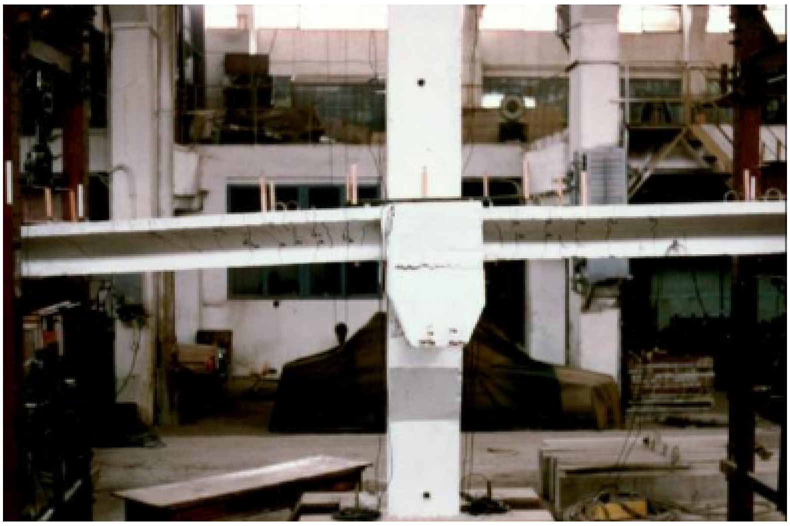

Figure 10 Condition of the test unit following testing to the Serviceability Limit State

4.2 **Experimental joint behaviour at the Damage Control Limit State**

Following testing to the Serviceability Limit State, a 10th semi-cycle (stage 4) was completed. For this cycle a constant axial force on the column of 800 kN was maintained and forces of P_A = 115 kN and P_B = 120 kN were applied at the longitudinal beam ends. These maximum force values corresponded to equivalent beam end displacements of $\Delta_A = 102.1\,\text{mm}$ and $\Delta_B = 69.7\,\text{mm}$.

The condition of the test unit at the end of the 10th semi-cycle is shown in Figure 11.

Figure 11 Details of the tested joint at the Damage Control Limit State

In the 10th semi-cycle the crack widths in the longitudinal beam reached up to 8 mm, indicating significant yielding of the longitudinal reinforcement. Also, spalling of cover concrete developed at the longitudinal beam-joint interface for negative beam moments. At the interface between the fibre concrete and the precast elements, crack widths in the SFRC reached values of 8 mm on the upper surface and 2 mm on the lower surface. Cracking due to shear also developed in the longitudinal beams.

The hysteretic response of the longitudinal beams are shown in Figure 12, indicating that beam A formed a negative moment plastic hinge and beam B formed a positive moment plastic hinge. The overall response of the test unit is shown in Figure 13, plotting the column (base) shear force V (see Fig. 7) against the measured joint rotation. The experimentally determined ductility 1 joint rotation of 0.00652 rad. was found by extrapolating the averaged measured rotations at $V = \pm 3/4V_n$. From Figure 13 it may be established that the joint was subjected to negative rotations corresponding to

$\mu_{?}$=1.4, and that for positive rotations the joint was loaded to approximately $\mu_{?}$ = 2.6. The calculated moment-curvature response of the longitudinal beam is shown in Figure 14.

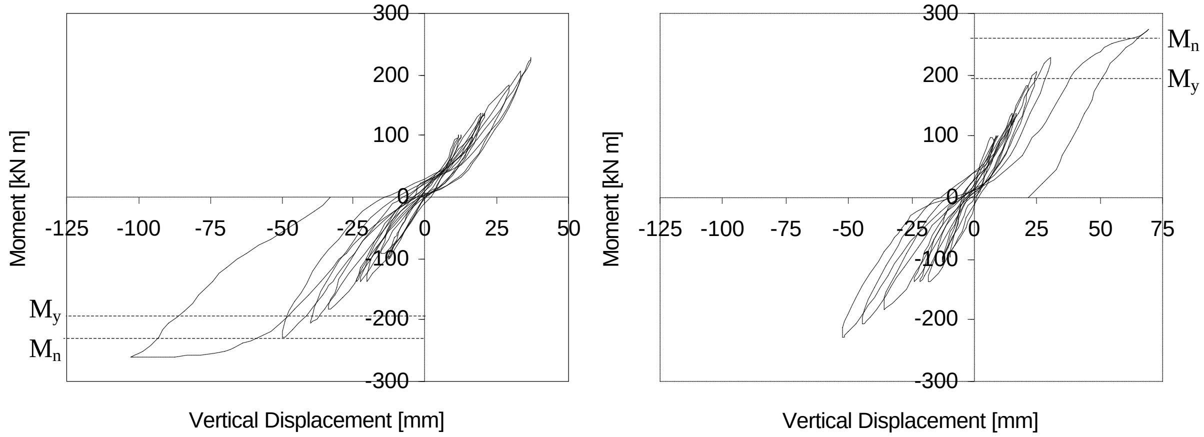

Figure 12 Moment-displacement histories for double-tee beams joint

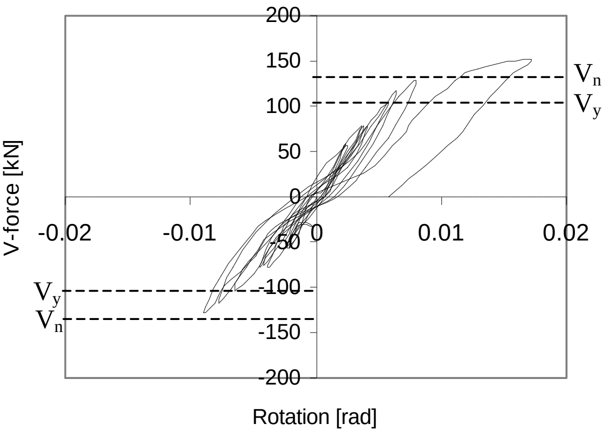

Figure 13 V-force-displacement histories for double-tee beams joint

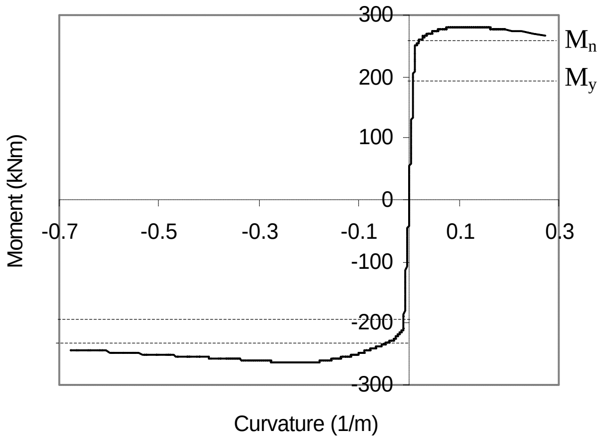

Figure 14 Moment-curvature histories for double-tee beams joint

From the Figure 13 it is evident that the test unit exhibited inelastic response without strength degradation. However, the test was terminated without further loading to avoid concerns about potential test unit failure, and because the original objective of the study, which was to conduct comparative testing with respect to a previous joint not having SFRC, had been satisfactorily investigated. Because of this premature conclusion to the test, it was not possible to determine the ultimate displacement capacity of the test unit.

5 CONCLUSIONS

This test was conducted in order to compare the performance of cast-in-place SFRC and cast-in-place non-fibrous concrete.

Performance of the test unit indicated that the set-up was satisfactory.

The test unit exhibited good behaviour. This statement is supported by the obtained experimental results:

? A displacement ductility ratio of $\mu_{\Delta} = 2.63$ was reached without loss of strength.

? Recorded ductility was due to widely spaced flexural cracking in the longitudinal beams, representing formation of competent plastic hinges.

? Crack dimensions were smaller than when using non-fibrous cast-in-place concrete.

? Test unit strength was in excess of that required, and beam overstrength of approximately 1.37 was observed.

The results of this study emphasised the superior behaviour of precast elements joined with cast-in-place SFRC. It is suggested that utilisation of this kind of concrete should not be limited to certain critical zones of frame structures, such as potential plastic zones beams, but may be extended to use in zones having a high risk of plastic behaviour, such as diaphragms.

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