EXPERIMENTAL SEISMIC RESPONSE OF REINFORCED CONCRETE COMPOSITE GIRDERS

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SUMMARY

The paper presents experimental researches carried out to clarify some aspects in seismic behaviour of reinforced concrete composite beams. The focus was on plastic-hinge zone and on shear resisting mechanism at the contact surface. The parameters taken into consideration during the tests were the horizontal shear stress level at the interface and the amount of the transverse reinforcement (connecting ties).The experimental models (simply supported beams, loaded at mid-span and exterior beam-column connection subassemblies) were detalied according to Romanian design code STAS 10107/0-90, in which the "shear-friction" concept is adopted to calculate the horizontal shear strength at the interface (similar to ACI 318/89 and NZS 3101/95). The experimental results have shown that, for high values of the horizontal shear stresses the Romanian code prescriptions concerning the interface slippage are correct, so reducing the transverse reinforcement leads to an unsatisfactory behavior.

INTRODUCTION

Reinforced concrete composite beams (prefabricated beams with cast-in place topping) are often encountered to framed structures in Romania, even in high seismic hazard area.

The seismic behavior of such elements, especially their plastic hinge zones, is not entirely understood. The specialists do not agree on:

- taken into consideration the shear-friction concept for plastic hinge zones;

- the length of this zone;

- the equivalent friction coefficient value;

- the need for inclined connection reinforcement;

- the safety of tension reinforcement anchoring by embedment in the thin (eventually cracked) layer of top concrete;

- the effectiveness of tension reinforcement anchoring in the thin (eventually cracked) layer of top concrete.

The paper presents researches aimed to clarify some of these problems.

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TESTING PROGRAMM

The experimental models were “extracted” from a R/C framed structure designed for a high seismic risk area. Three types of models has been studied (G4, G5 and G6, Fig. 1), corresponding to a horizontal shear stress of 1.5, 2 and respectively 3 times the design tensile stress of concrete. Fourteen specimens have been tested.

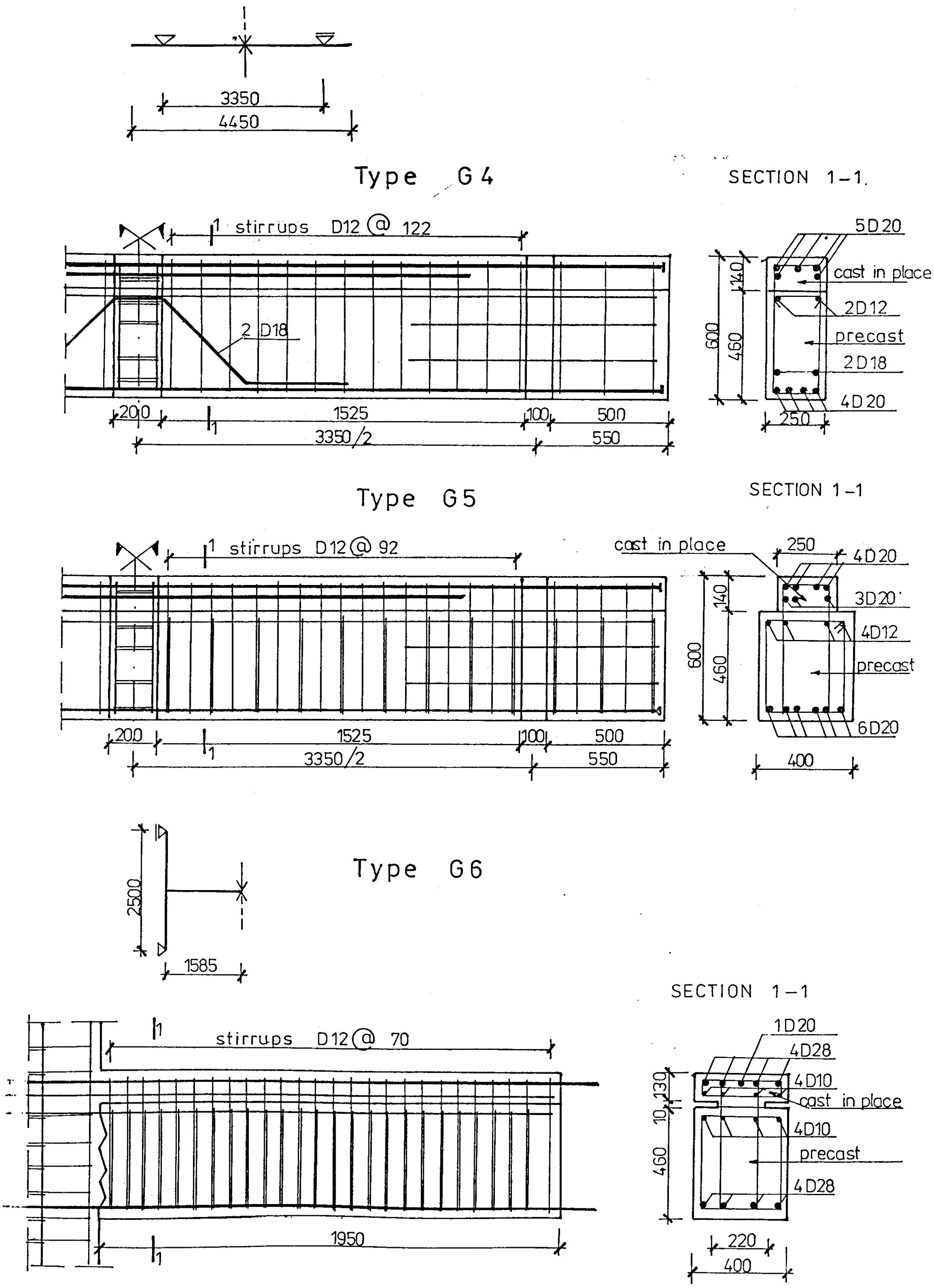


Figure 1. Test specimens

The upper face of the precast element (the contact surface) was clean, but not intentionally roughened and not free of laitance. A value of 0.7 for the equivalent friction coefficient was taken, in accordance with the Romanian design code STAS 10107/0-90 [4]. The concrete topping was cast 14 days after the pouring of the precast element.

A modified variant of G5 and G6 type (named G5M and G6M respectively), with the amount of connecting transverse reinforcement reduced by 50% was also studied. This because previous researches [2] have shown that, for elements with the upper surface similar to G4, G5 and G6, a value of 1.4 for equivalent friction coefficient can be taken if the design horizontal shear stress equals the design tensile strength of concrete.

The effective strength of materials were:

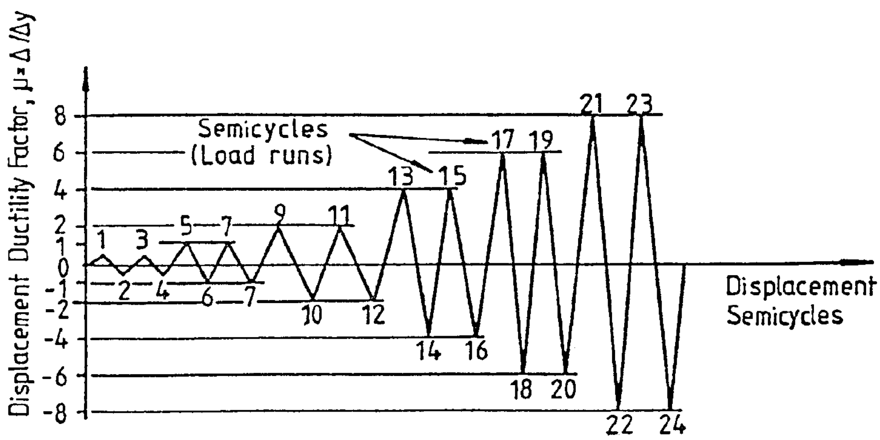
- steel: 405 MPa yielding strength and 620 MPa tensile strength;

- concrete: 23 – 29 MPa compressive cubic strength in precast concrete and 20 – 26 MPa in topping concrete.

The experimental models were simply supported beams loaded at midspan and beam-exterior column connections.

The testing forces were so applied, not to affect the local behaviour of the composite beam.

The tests were carried out in post-elastic range through seismic type cyclic reversed loading, with displacement control. The loading history is presented in Fig. 2.



Δ - curent displacement

Δ_y – displacement corresponding to bending reinforcement yielding

Figure 2. Load history

TEST RESULTS

The first transverse (flexural) cracks appeared under forces that were only 20 – 35 % from the yielding force. The first longitudinal interface cracks appeared under yielding force (or yielding displacement Δy).

The elements G4, G5 and G6 cracked all along the interface under quite high loading, corresponding to 4 – 6 Δy. Such a crack occurred at elements G5M and G6M much earlier, under loading corresponding to 2 Δy.

The 0.1-mm interface slippage at G4, G5 and G6, together with the bar slippage, occurred under the loading corresponding to a displacement of 4 Δy, while for G5M and G6M such a slippage occurred for 1 – 2 Δy.

In the post-elastic range the strains and stresses in topping concrete reinforcement were different – less in short bars than in continuous bars. The difference increased with the load, attaining, for stresses, 15 – 20 % at 4 Δy.

In what G5M and G6M are concerned, the strains in topping concrete reinforcement did not increased when the load exceeded the value corresponding to 2 Δy (Fig. 3). This was obviously a sign that beyond that limit the parts of the composite section did not act together any more.

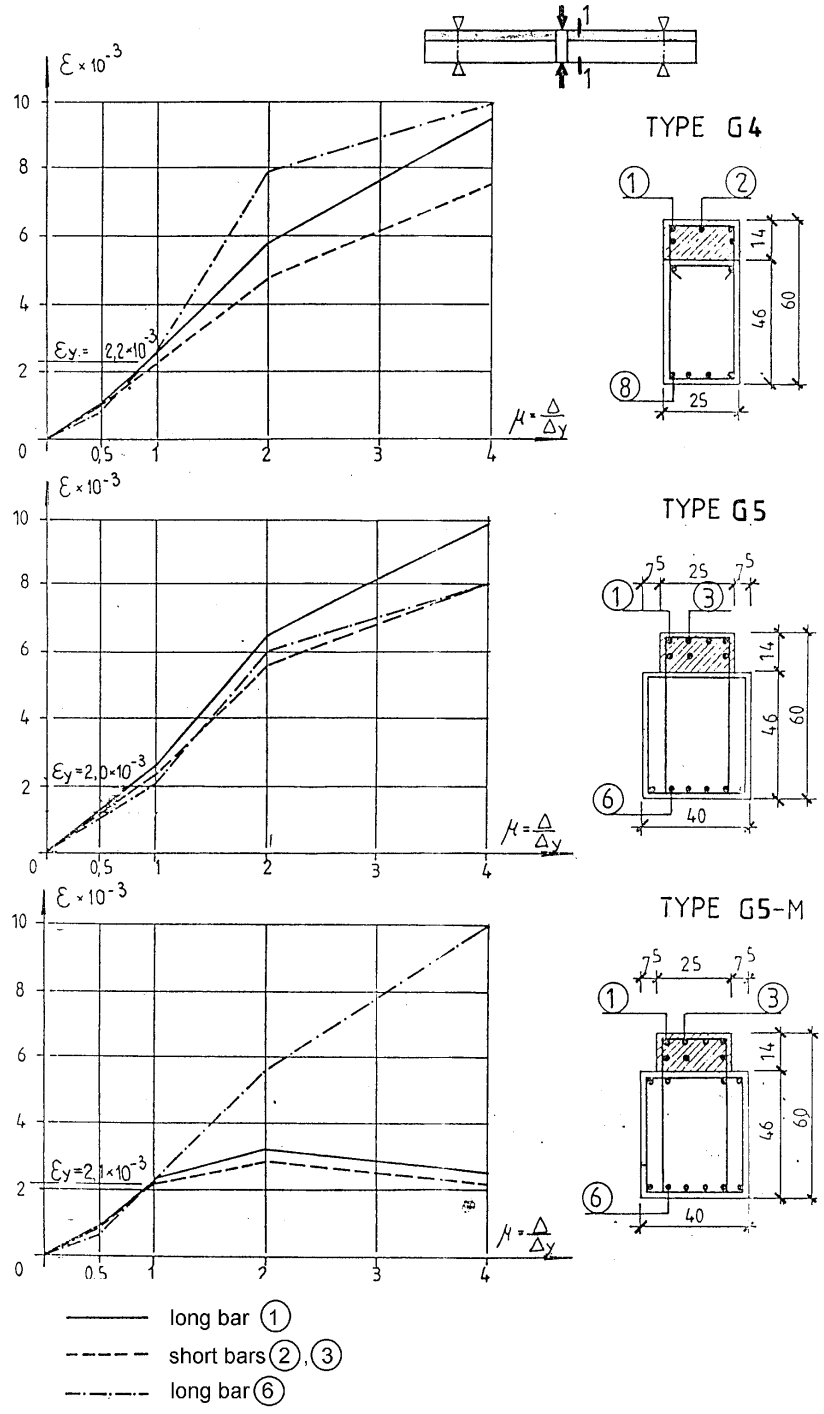


Figure 3. Tensile strains in longitudinal bars

The stresses in connecting ties, which yielded at 4 Δy, exceeded with 50 % those, measured in precast element ties.

The experimental forces corresponding to imposed displacements increased continuously, so at 4 – 6 Δy exceeding with 50 – 60 % (at G5) those for 1 Δy.

This force increasing was observed, at the elements with reduced connecting ties, only up to 2 Δy. After that, the force decreased (for G5M with 25 % at 4 Δy compared to that for 1 Δy). That happened when the interface was in the tension zone.

The maximum flexural moment experimentally obtained to the calculated (on the basis of steel and concrete actual strength values) one ratio was, for the loading which put in tension the cast-in-place concrete, with 20 % less than in the case with the cast-in-place concrete in the compression zone. A possible explanation could be a not fully participation of the short bars from the cast-in-place concrete.

The element ductility factor obtained was fairly good, attaining values about 8, excepting G5M and G6M (ductility factor nearly 6).

The elements G4, G5 and G6 had a good load-displacement response, showing ability to dissipate energy. One could not say the same about G5M and G6M, which shown an asymmetric response, affected by pinching (Fig. 4).

Type G4 Type G5 Type G5-M

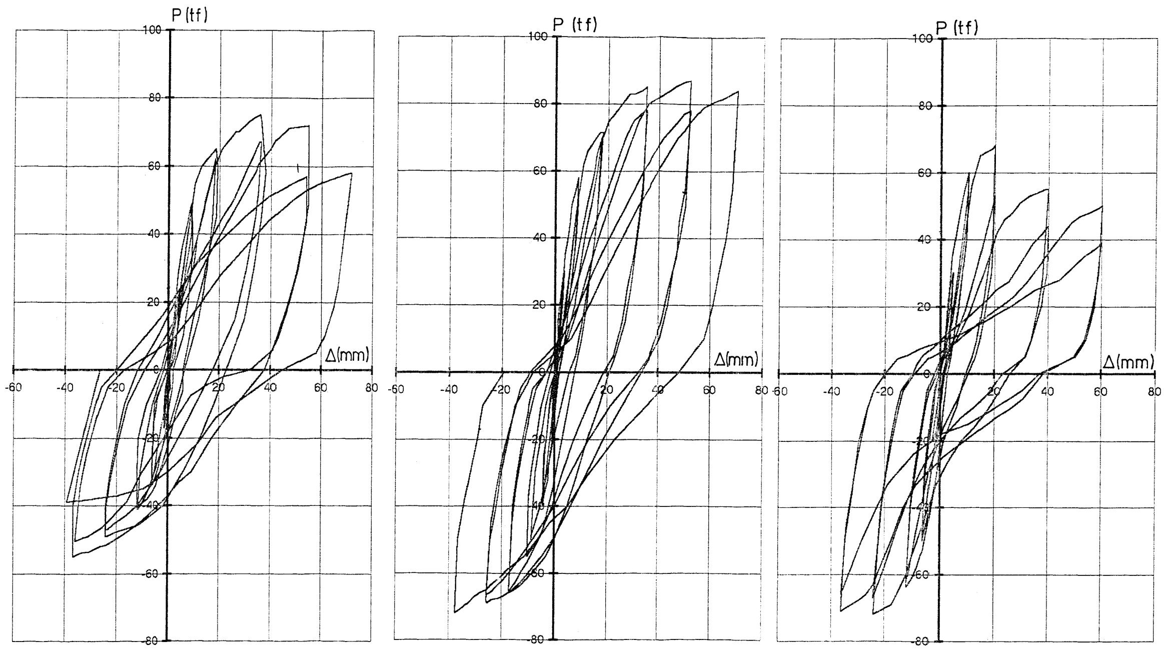
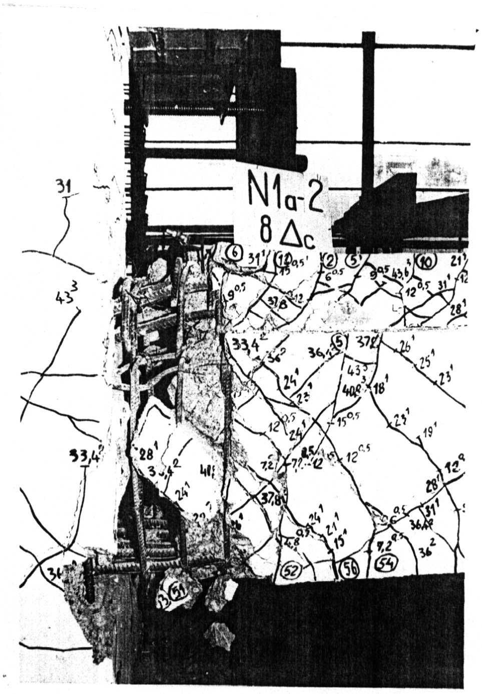


Figure 4. Load-displacement response

The failure mechanism (inclined shear failure for G4, flexural failure for G5 and G6 – Photo 1) was similar to that of fully cast-in-place element. The acting together for G5M and G6M deteriorated beyond 1 Δy quite rapidly, at first through bond failure and slippage occurrence all along the interface, then through the pull-out of the reinforcement from the cast-in-place concrete, the crumbling of the cast-in-place concrete and finally by the crushing of the concrete in the precast beam (Photo 2).



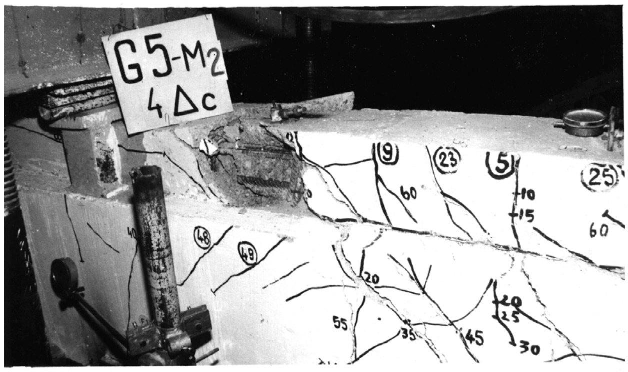


Photo 1 **Photo 2**

The length of the plastic hinge zone was determined as the zone along which the yielding of the longitudinal reinforcement occurred. It increased with the loading, from 0.5 – 0.7 h (h being the height of the cross section) for 1 Δy, till 1.5 Δy for 4 Δy (G6) or 6Δy (G4 and G5). The length of the plastic hinge zone for G5M and G6M did not increased beyond 0.75 h (for 4 Δy).

CONCLUSIONS

The results have shown that for studied composite elements, detailed from the point of view of anchorage length according to Romanian standard, short bars in the cast-in-place concrete were not so effective as continuous ones.

The experimental models detailed as required by Romanian standard, developed an appropriate strength and behave properly at high reversed loading. A good behaviour was observed for G4, G5 and G6 models. The two parts of the composite section did act together, even in the yielding stage. A displacement ductility factor of 8 was obtained and a good hysteric behaviour was observed. Limited horizontal cracks at the interface were observed in the post-elastic range, but the failure mechanism was similar to that of fully cast-in-place element.

The G5M and G6M models, with reduced amount of connecting ties, have proved lower slip strength and a not very good behaviour at cyclic reversed loads. Horizontal cracks extended over the entire length of the interface in the post-elastic range, when the slippage at the interface occurred and greatly affected the model behaviour. The slippage of the tensioned bars in the cast-in-situ concrete also occurred.

It was proved also that, for elements designed for high intensity horizontal shear, one couldn't count on a value of 1.4 for the friction coefficient.

Further investigations are needed to clarify the supposed relation between the value of equivalent friction coefficient (in "shear-friction" concept) and the design intensity of the horizontal shear stress.

The absence of inclined connecting reinforcement did not affect the composite element behaviour. So, the Romanian code provisions compelling the use of such reinforcement in the case that horizontal shear stress exceeds twice the design tensile stress of the concrete should be verified through other experiments.

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