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ASSESSMENT OF SHEAR STRENGTH FOR EXISTING BRIDGES WITH LOW AMOUNTS OF SHEAR REINFORCEMENT

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Abstract

In this paper, the results of a testing campaign carried out at Ecole Polytechnique Fédérale de Lausanne on 2 reinforced concrete girders and 10 post-tensioned concrete girders (10 meters long, 0.78 m high) are outlined. The specimens are provided with very low amounts of shear reinforcement and some of them present defective stirrup anchorage to simulate realistic development conditions of existing bridge girders. The experimental results are discussed with reference to a number of design models to discuss on their accuracy and pertinence.

Keywords: Concrete Bridges, Post-Tensioning, Prestressed Girders, Shear Design

1 Introduction

Design of concrete girder bridges has significantly evolved during the last decades. This is particularly relevant with respect to shear design, motivated by changes in actions and design models. As a consequence, assessing the shear strength of existing bridges leads in many cases to unsatisfactory safety levels. Furthermore, many existing bridge girders do not comply with current code regulations with respect to minimum amounts of shear reinforcement. This situation can lead to expensive retrofitting and strengthening of a significant number of existing bridges. The assessment of the shear strength of these members has thus become a significant task for structural engineers.

Design codes are not always appropriate for assessing the strength of existing bridges. They propose safe models providing sufficient accuracy for design of a wide number of structures. However, these models do not account for some properties of prestressed bridges and neglect a number of shear-transfer actions that can be relevant for their strength. This is typically the case of shear carried by the inclination of the compression chord, the increase of the stress in the tendons or the effective strength of concrete in the web of cracked members (Fernández Ruiz and Muttoni, 2008). Developing more realistic approaches for evaluating these parameters may significantly increase the estimated strength of a structure with respect to code provisions, avoiding in many cases unnecessary retrofitting or strengthening.

In the present paper, this topic is investigated by means of a test series on 12 concrete beams where the main parameters are the shape of the cross section, the amount of shear reinforcement, the post-tensioning force and the anchorage of the stirrups. The test campaign is described in the following as well as a comparison of its results to different design models discussing on their accuracy.

2 Test series

2.1 Test setup and dimensions of the test specimens

The test setup was designed to represent in a realistic manner the inner forces of continuous bridges (with clamping moment over supports), see Figure 1. The central part of the girder (150 mm thick) corresponds to the testing region, the rest of the specimen having a larger thickness (400 mm) to avoid shear failures. During testing, continuous measurements of the load, deflections and rotations of the member were recorded. In addition, demec measurements were also performed at selected load-stages to investigate on the kinematics of the web and flanges at failure.

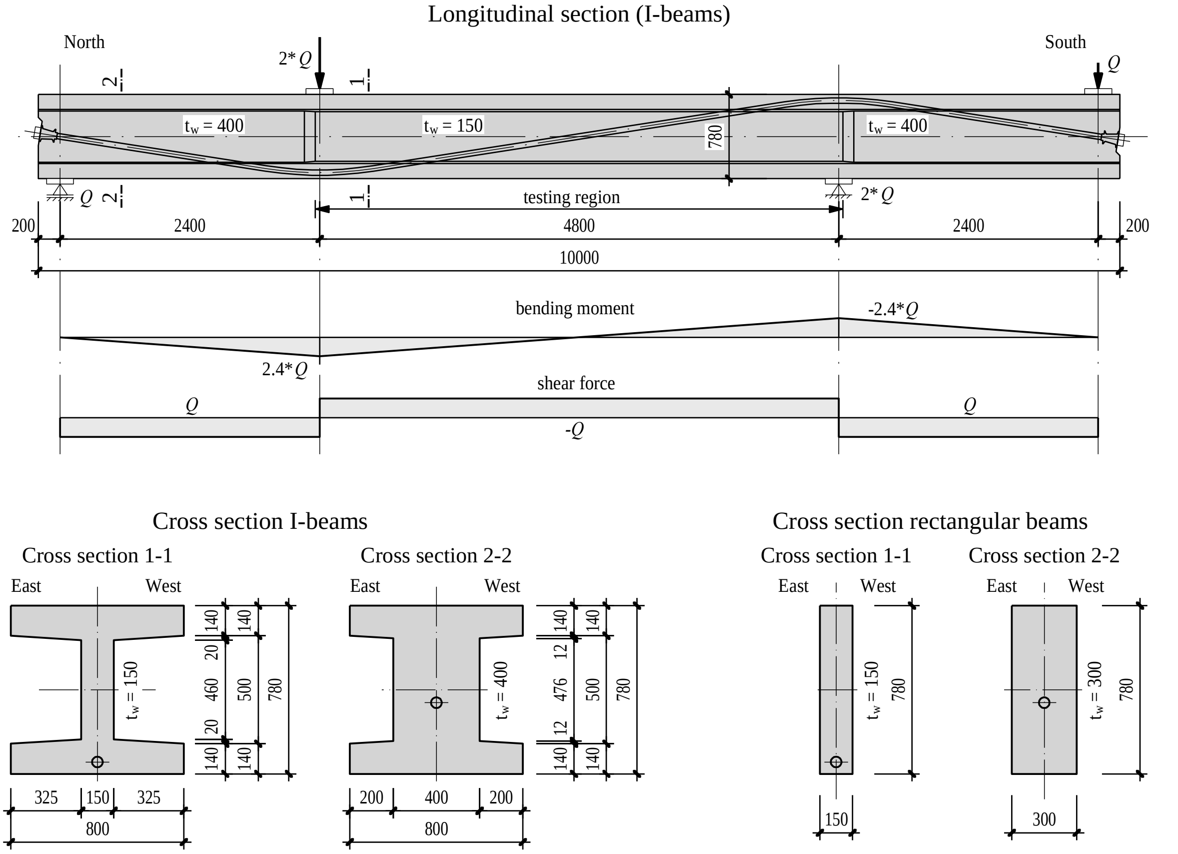

Fig. 1 Test setup, bending moments, shear forces and dimensions of the test specimens (dimensions in [mm]).

2.2 Test specimens and main parameters

The main parameters of the test series are the shape of the cross section (rectangular or flanged), the amount of post-tensioning (P/A), the geometrical shear reinforcement ratio (ρ_w) and the stirrup development conditions as indicated in table 1.

Table 1: Parameters of the test series

Beam SR..	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**	**32**
cross-section	I	I	I	I	I	I	I	I	I	I	▯	▯
P/A [MPa] *	2.5	2.5	2.5	2.5	5.0	5.0	5.0	-	2.5	2.5	3.0	-
ρ_w [%] *	0.09	0.13	0.06	0.25	0.09	0.06	0.19	0.09	0.25	0.25	0.09	0.09
stirrup *	▯	▯	▯	⊔	▯	▯	▯	▯	⊔	⊔	▯	▯
$A_s = A_s'$ [mm^2]	4279	4279	4279	4279	3223	3223	3223	3223	4279	1847	1219	2124
f_c [MPa]	30.8	33.7	35.3	31.3	33.1	36.9	28.3	37.8	29.8	31.4	31.3	35.2
f_{yw} [MPa]	585	585	585	575	585	585	575	585	575	585	525	525

(*) The Parameters refer to the central part of the beam. P = nominal post-tensioning force; A = cross section area in the testing region.

The geometrical dimensions (span, cross-section…) are the same for all beams with and without flanges (refer to Fig. 1). The shear reinforcement in the testing region consists of single bars or stirrups 6 mm diameter leading to a geometrical reinforcement ratio ρ_w between 0.09% and 0.25%. The maximum stirrup spacing is 300 mm. Ten beams are post-tensioned by means of one or two tendons grouted in steel ducts after post-tensioning. The longitudinal reinforcement consists of straight bars of ordinary reinforcement with diameter ranging between 10 mm and 26 mm. The total area of longitudinal reinforcement (A_s) is constant over the whole length of the beam and is the same for the top and bottom flanges. The cross-section area of this reinforcement is indicated for each specimen in Table 1.

Normal strength concrete was used for all specimens with a maximum aggregate size of 16 mm. The shear reinforcement in the testing region of the beams with flanges was cold-worked presenting a strain at tensile rupture ε_u = 3.0% and a tensile strength-to-yield strength (f_{tw}/f_{yw}) equal to 1.08. The corresponding values for the two beams without flanges are ε_u = 5.5% and f_{tw}/f_{yw} of 1.12. The yield strength of the shear reinforcement f_{yw} as well as the concrete cylinder strength f_c at the day of testing are provided in Table 1.

3 Test results, discussion and comparison with different models

3.1 Test results and discussion

The ultimate shear strength $V_{R,test}$ of all beams is indicated in Table 2. All girders failed in shear. The beam SR31 could be externally reinforced and tested a second time which leads to the value of SR31B. Over the whole test series three different failure modes can be observed (refer to Figs. 3,4).

Table 2: Measured shear strength of the tested specimens $V_{R,test}$

Beam SR..	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**	**31B**	**32**
$V_{R,test}$ [kN]	399	459	364	579	484	457	606	222	585	581	309	303	173

The first failure mode applies for the beams with flanges and low amounts of shear reinforcement (as specimen SR21). After reaching the maximum load, the beams show a softening behaviour, with large crack openings and rupture of the stirrups at the cracks, followed by a stable residual resistance of about 60 to 70% of the ultimate strength. The second failure mode applies for the girders with flanges and a higher amount of shear reinforcement (as specimen SR24). The shear force – deflection curve looks the same as for the beams with low amounts of shear reinforcement with a residual resistance of about 60 to 70% as well and a large deformation capacity. Contrary to the other specimens, spalling of the concrete along the tendon axis can be observed. The third failure mode applies for the beams without flanges (as specimen SR31). The beams show brittle failures, localizing strains in a single crack with almost no residual strength.

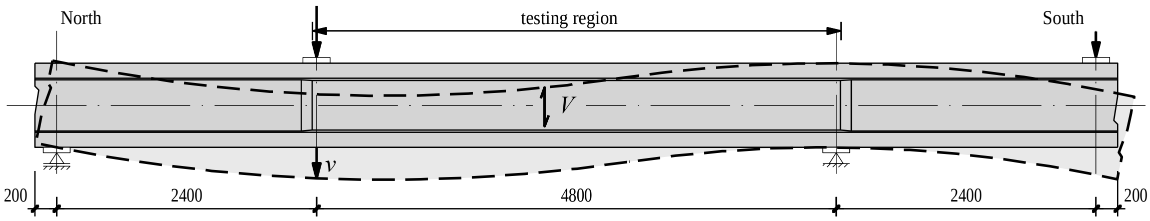

Fig. 2 Measured deflections (dimensions in [mm]).

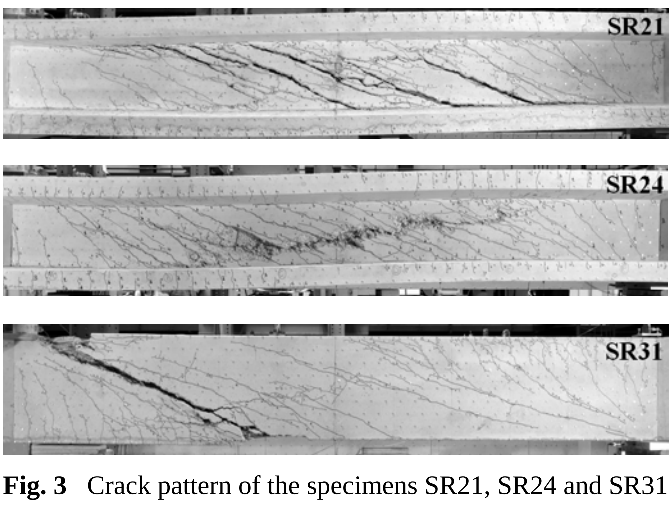

Fig. 3 Crack pattern of the specimens SR21, SR24 and SR31 in the testing region (after testing).

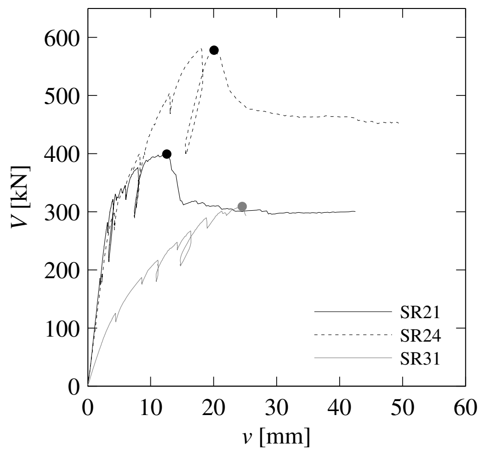

Fig. 4 Shear force (V) versus deflection (v) of the specimens SR21, SR24 and SR31.

The test series shows that the shear reinforcement ratio and the amount of post-tensioning have a significant influence on the ultimate strength of all tested beams and on the rupture mode of the flanged girders. Also, the role of flanges seems positive in order to increase the strength of the member as well as the deformation capacity at residual strength.

3.2 Comparison of test results to design models

In order to check the accuracy of different design models, a comparison of the ultimate shear strength of the tested beams with the predicted shear strength of Eurocode 2 (CEN, 2004), Model Code 2010 – Level III (*fib*, 2010), Response-2000 (Bentz, 2001), and the elastic-plastic stress field method (EPSF) (Fernández Ruiz and Muttoni, 2007) was performed. The results are given in Table 3. For all models, a reduced web with of 125 mm was considered instead of the effective web with of 150 mm. This takes into account the reduction for the presence of a steel duct in the web, as described by Fernández Ruiz and Muttoni (2008). Eurocode 2 leads to the most conservative predictions while MC2010 and Response 2000 allow improving the estimate of the strength with a similar scatter of results. The best results are obtained by using the EPSF method, with an average value of 1.06 and a coefficient of variation of 0.05.

Table 3: Comparison of shear strength to design models – $V_{R,test}/V_{R,model}$

Beam SR..	21	22	23	24	25	26	27	28	29	30	31	31B	32	Avg	CoV
EC2	1.58	1.45	1.68	1.13	1.46	1.54	1.23	1.63	1.14	1.13	1.72	1.69	1.36	**1.44**	**0.16**
MC2010	1.15	1.17	1.09	1.14	1.09	1.05	1.15	1.02	1.16	1.26	1.31	1.29	0.91	**1.14**	**0.10**
Response	1.15	1.16	1.08	1.16	1.05	1.06	1.11	1.00	1.17	1.16	1.29	1.27	1.02	**1.13**	**0.08**
EPSF	1.08	1.07	1.03	1.03	1.03	1.03	1.04	1.01	1.04	1.08	1.17	1.14	0.99	**1.06**	**0.05**

4 Conclusions

This paper presents the results of a test series on post-tensioned concrete girders with low amounts of shear reinforcement and compares them to a number of design approaches. The main conclusions of the investigation are:

1) The amount of post-tensioning and the shear reinforcement ratio have a significant influence on the strength and failure modes developed.
2) Two failure modes are observed for flanged beams (failure of the entire web with several wide cracks developing on it or localized crushing of concrete with cover spalling along the tendons), depending on the level of post-tensioning and on the shear reinforcement ratio.
3) The deformation capacity and the post-peak resistance of flanged girders are significantly higher than for beams without flanges. The presence of flanges is thus shown to be beneficial.
4) A comparison of the test results with different model predictions shows that design models provide in general conservative estimates for the tested beams and that the elastic-plastic stress field approach leads to the most accurate predictions.

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