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Moment–shear interaction of stiffened plate girders—Tests and numerical model verification

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ABSTRACT

Results of four full-scale tests on plate girders stiffened with transverse and longitudinal stiffeners subjected to interaction of high bending moment and shear force are presented and discussed. In longitudinal direction the web was stiffened with open or closed stiffeners positioned in the compression zone. Detailed information on initial geometric imperfection and residual stresses is given. The experimental results were used to verify numerical model. The resistance is compared against reduced stress method and effective width method given in EN 1993-1-5.

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1. Introduction

Steel plate girders are commonly used to support vertical loads over long spans, where bending moment and shear force exceed the capacity of standard hot rolled beams. They are usually composed of compact flanges, slender webs, transverse and longitudinal stiffeners. At internal support of continuous plate girder web panels are subjected to combination of high shear force and high bending moment.

Basler [1–3], was the first who proposed mechanical model to take into account post-buckling resistance of the web plate subjected to the interaction of shear force and bending moment. Assuming that shear is carried only by the web, the maximum shear resistance is reached when the tension field in the web is fully utilized. The shear resistance of the web plate is independent of the bending moment as long as the moment is less than bending capacity of flanges alone. When higher bending load is applied, a part of the moment is also resisted by the web, which reduces the shear resistance. The model proposed by Basler was developed only for longitudinally unstiffened girders. Further design models were based on Basler's assumption with modification of the shear resistance of the web plate and modification of the interaction model. The interaction of bending moment and shear force is covered by the following authors: Herzog [4], Porter et al. [5], Bergfelt [6], Ostapenko and Chern [7], Škaloud and Rockey [8], Höglund [9–11] and Fujii [12,13].

For each of the above mentioned methods the ultimate shear resistance is defined with modified collapse mechanism. The interaction of bending moment and shear load is defined with linear expression (in case of [4,6,12]) or with quadratic expression as in case [5] and [7–9].

At the ultimate limit state the longitudinal stiffeners are accounted only for Aarau method, Cardiff method and Prague–Cardiff method. All other methods were developed for longitudinally unstiffened girders.

Most of the experimental tests on plated girders have been performed out of the domain of the bending–shear interaction in the web plate (high shear force and high bending moment). The tests on longitudinally stiffened girders with information on M–V interaction in the web panel are gathered in Table 1. The test specimens of Schueller and Ostapenko [14] were stiffened with double sided longitudinal stiffener in the compressed part of the web. The stiffener was designed to prevent global buckling of the whole panel. Evans [15] and Public Work Research Institute [16] performed tests on girders stiffened with one-sided stiffeners in the compression part. The global buckling of the panel was observed within all 5 tests.

The M–V interaction model given in EN 1993-1-5 [17], which is based on slightly modified Basler approach, was verified for longitudinally unstiffened girders by Veljkovic and Johansson [18], but the information on the M–V interaction behaviour of longitudinally stiffened girders is not available in the literature.

In EN 1993-1-5 the same interaction model is used for longitudinally stiffened girders, but the behaviour of such girders can be different from that of unstiffened girders. This gap in knowledge was the main motivation to start this research work.

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Table 1
Experimental tests on longitudinally stiffened girders that contain information on bending–shear interaction in the web.

Test	a [mm]	h_w [mm]	t_w [mm]	b_f [mm]	t_f [mm]	$M_{exp.}/M_{th.}$	$V_{exp.}/V_{th.}$
Schueller & Ostapenko [14]							
UG 5.2	1397	1217	3.02	254	19.20	1.05	1.05
UG 5.3	1778	1217	3.02	254	19.20	1.21	1.21
UG 5.4	2159	1217	4.65	254	19.20	1.19	1.19
UG 5.5	1016	1217	4.65	254	19.20	1.02	1.02
Evans [15]							
PB1	750	1008	4.40	300	15.10	0.93	1.11
PA1	750	1008	3.83	300	15.10	1.03	1.03
Public Work Research Institute, Japan [16]							
C-26	1000	1650	4.73	250	12.12	1.03	1.03
C-27	1000	1650	4.73	250	12.12	1.04	1.04
C-28	1000	1650	4.73	250	12.12	1.00	1.00

Table 2
Results from tensile coupon-tests in plates.

Plate	$R_{p\ 02}$ yield stress [MPa]	R_m ultimate stress [MPa]	f_u/f_y	Average reduction of $R_{p\ 02}$ [%]	Static yield stress [MPa]
5 mm	385	539	1.40	7.19	357
6 mm	405	539	1.33		376
7 mm	391	561	1.44		363
8 mm	399	552	1.38		371
10 mm	395	542	1.37		367
15 mm	369	520	1.41		342
20 mm	375	543	1.45		348
22 mm	354	536	1.52		328

2. Experimental work

2.1. General

The aim of four full scale tests was to examine the characteristics of longitudinally stiffened plated girders under high bending and shear load. Further on, the test results also serve for the verification of the numerical model.

The tests were performed on two girders stiffened with transverse and longitudinal stiffeners. On each of them two panels were tested. The girders were built up of symmetric cross-section in the first case, and of unsymmetric cross-section in the second one. The panel aspect ratio was set to 1.0 for panels stiffened with longitudinal open stiffener and 1.5 for girders stiffened with closed cross-section. The intermediate transverse stiffeners which divided the girder into several panels were designed as rigid to prevent any interaction between adjacent panels. Analytical model given in [19,20] was used to fulfil the EN 1993-1-5 [17] resistance as well as stiffness criterion. The system length of the girder was carefully defined in order to obtain the proper ratio of bending moment and shear force in the tested panel. The relative bending stiffness γ of longitudinal stiffeners was chosen so that the shear buckling resistance of the subpanel was decisive. (The critical shear stress of the subpanel was several times smaller than the critical shear stress of the stiffened panel, $\tau_{cr,\ sub\text{-}panel} \ll \tau_{cr,\ panel}$). In each test the stiffeners were positioned in the compression zone of the panel. Basic testing parameters for all four test panels are listed below:

- Symmetric cross-section stiffened with open stiffener (*SO*)

$$h_w/t_w = 214, \alpha = 1.0, \gamma = 41.55$$

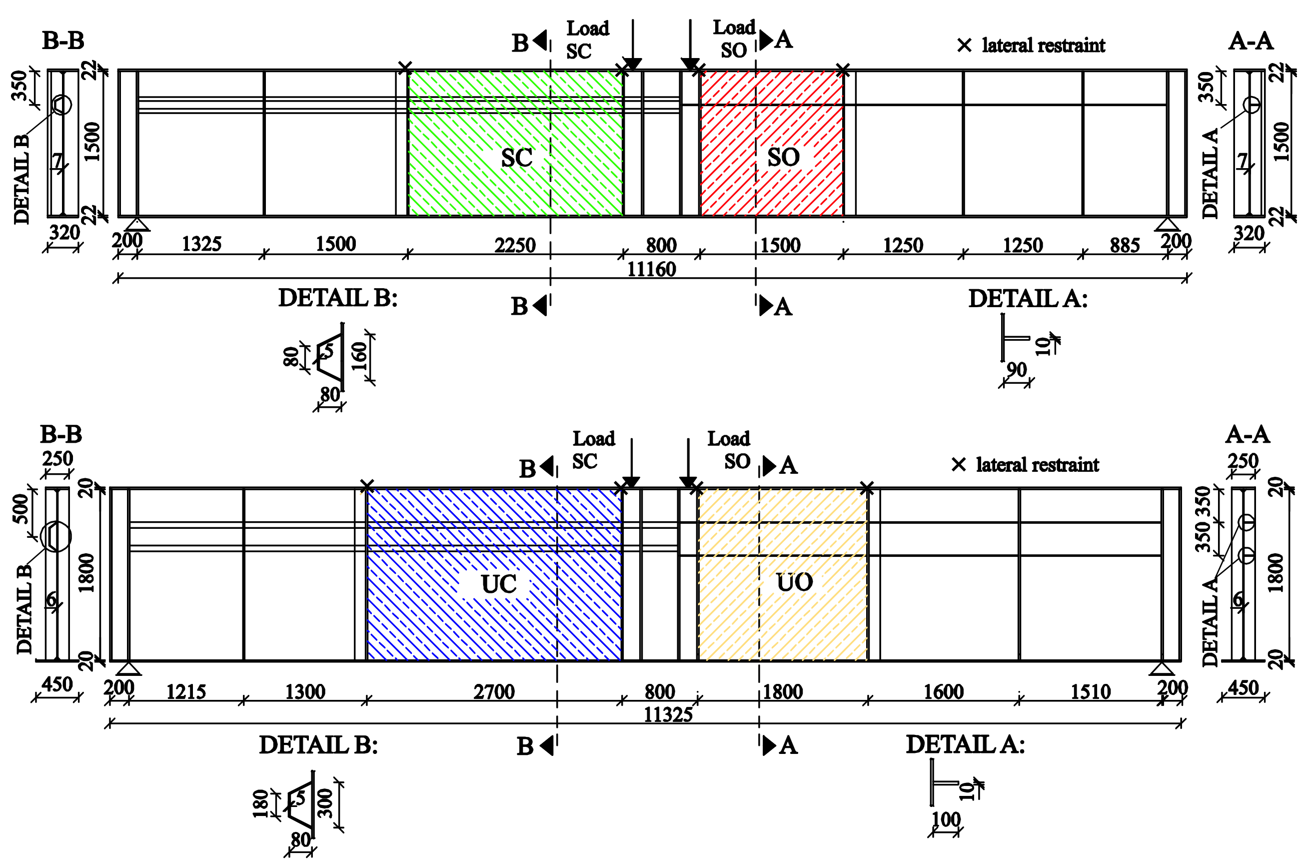

Fig. 1. Test girder geometry.

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