

Title:	Shear-Fatigue Behaviour of RC Cantilever Bridge Deck Slabs under Concentrated Loads
Authors:	Natário F., Fernández Ruiz M., Muttoni A.
Published in:	fib Congress
Pages:	9 p.
City, country:	Copenhagen, Denmark
Year of publication:	2015
Type of publication:	Peer reviewed conference paper

Please quote as:	Natário F., Fernández Ruiz M., Muttoni A., <i>Shear-Fatigue Behaviour of RC Cantilever Bridge Deck Slabs under Concentrated Loads</i> , fib Congress, Copenhagen, Denmark, 2015, 9 p..
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SHEAR-FATIGUE BEHAVIOUR OF RC CANTILEVER BRIDGE DECK SLABS UNDER CONCENTRATED LOADS

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Abstract

Shear has been observed to be the governing failure mode of RC cantilever deck slabs of bridges without shear reinforcement when subjected to concentrated loads of heavy vehicles. This observation has been verified experimentally for a quasi-static application of the load. However, concentrated loads have a repetitive nature, causing loss of stiffness and potential strength reductions due to fatigue phenomena.

In this paper, the fatigue behaviour of cantilever bridge deck slabs is investigated. A specific experimental programme consisting on eleven tests under concentrated fatigue loads and four static tests (reference specimens) is presented. The results show that cantilever bridge deck slabs are significantly less sensitive to shear-fatigue failures than beams without shear reinforcement. Slabs that failed due to rebar fractures presented significant remaining life after first rebar failure occurred. The test results are presented and finally compared to the shear-fatigue provisions of the *fib*-Model Code 2010.

Keywords: shear strength, Model Code 2010, bridge deck slabs, concentrated load, fatigue behaviour

1 Introduction

Reinforced concrete cantilever bridge deck slabs without shear reinforcement are generally design/assessed under the effect of concentrated loads of heavy vehicles, which may cause shear, punching shear or flexural failures. Amongst these potential failure modes, shear is the most common governing failure mode under quasi-static application of the load (Vaz Rodrigues, Fernández Ruiz & Muttoni 2008; Rombach & Latte 2009; Reissen & Hegger 2013; Natário, Fernández Ruiz & Muttoni 2014).

The concentrated loads that simulate heavy vehicles have nevertheless a repetitive nature and may cause potential stiffness and strength reductions due to fatigue effects. Fatigue failure modes are the same as the static ones and can be governed by rebar fracture and failure of concrete. Fatigue testing on reinforced concrete slabs without shear reinforcement under concentrated loads has been mainly focused in the past on simply supported or inner slabs (Sawko & Saha 1971; Hawkins 1976; Batchelor, Hewitt & Csagoly 1978; Okada, Okamura & Sonoda 1978; Sonoda & Horikawa 1982; Perdikaris & Beim 1988; Perdikaris, Beim & Bousias 1989; Youn & Chang 1998; Toutlemonde & Ranc 2001; Graddy & al. 2002; Hwang & al. 2010). Table 1 presents some geometric properties of the slabs tested by these authors. In comparison with typical deck slabs of concrete bridges, it can be observed that several specimens have relatively low thicknesses (<100mm) and others have low reinforcement ratios ρ (<0.6%) or fairly large ones (>1.5%). In addition to these facts, simply supported slabs present a potentially different mechanical behaviour than cantilever slabs (Natário, Fernández Ruiz & Muttoni 2014). These facts point out the fact that available testing is not necessarily representative of the actual behaviour of cantilever deck slabs of bridges.

With respect to specific testing on beams, a number of three and four-point bending tests have been performed in the past on reinforced concrete beams without shear reinforcement. An extensive summary on this topic can be found elsewhere (Gallego, Zanuy & Albajar 2014). Beams can fail in bending or shear in both static and fatigue tests (bending failures being associated to rebar fracture or

concrete crushing). Shear-fatigue failures were first studied by Chang & Kesler (1958a-b), who have observed the so-called *diagonal-cracking* failures, where failure takes place with the formation of a diagonal shear crack, and the *shear-compression* failures, where the formation of a diagonal shear crack does not imply the member collapse. Failure only takes place in the latter case when the propagation of the shear crack reduces the depth of the compression zone to an extent such that it can no longer resist the acting compressive forces.

Beams, unlike cantilever slabs subjected to concentrated loads, do not exhibit a two-way action (Fig. 1) and consequently cannot redistribute its internal forces due to bending and shear cracking (Natário, Fernández Ruiz & Muttoni 2014). Moreover, the ratio between the maximum applied moment m_{max} and the maximum applied shear force v_{max} in cantilever slabs at the support is lower than for cantilever beams (Rombach & Latte 2009), refer to Fig. 2.

Table 1
Properties of fatigue tested slabs under concentrated loads

Tests	ρ [%]	thickness [mm]	spanning	spans [cm]	widths [cm]	type {1}
Sawko & Saha (1971)	-	38	one-way	229-152-114	152	PC
Sawko & Saha (1971)	1.7	76	one-way	114	152	RC
Hawkins (1976)	1.3	127	one-way	122	127	RC
Batchelor, Hewitt & Csagoly (1978)	0.0-0.2-0.4-0.6	22-18-12	one-way	30	305	RC
Okada, Okamura & Sonoda 1978	1.1-1.3	170-180	two-way	235-360	-	RC
Sonoda & Horikawa (1982)	1.3	60	two-way	80-250	-	RC
Perdikaris & Beim (1988)	0.0-0.3-0.7	32	one-way	32	230	RC
Perdikaris, Beim & Bousias (1989)	0.0-0.3-0.4-0.7	72	one-way	71	170	RC
Youn & Chang (1998)	1.0	60	one-way	70	210	RC
Toutlemonde & Ranc (2001)	1.2	180	one-way	250	500	RC
Graddy & al. (2002)	3.2	191	one-way	183	213	RC
Hwang & al. (2010)	-	115	one-way	270	430	PC

{1} RC – reinforced concrete; PC – prestressed concrete

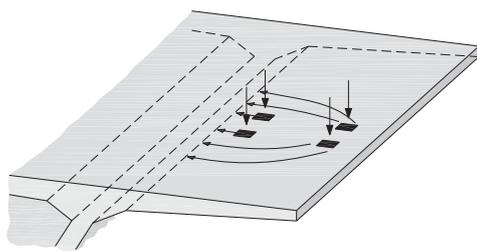


Fig. 1 Shear transfer mode in a cantilever slab subjected to concentrated loads

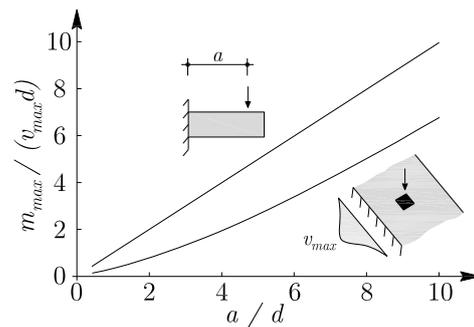


Fig. 2 Comparison between the bending to shear ratio of a beam and a cantilever slab subjected to a concentrated load (Rombach & Latte 2009) (d – flexural depth)

To the authors' knowledge there are no fatigue tests on specimens that represent typical cantilever slabs under concentrated loads. In order to provide such experimental evidence, an experimental campaign has been performed at the Ecole Polytechnique Fédérale de Lausanne (Switzerland). Four static tests on two slabs (two per slab and load position) and eleven fatigue tests on eight slabs (four slabs per load position) have been performed. The specimens are full-scale slabs (3.00 m x 3.00 m x 0.25 m) with a central line support.

2 Test campaign

2.1 Test setup

The test setup is shown in Fig. 3. The slabs are centrally supported on an I-shaped aluminium profile equipped with 30 vertical strain gauges on each side of the web with a constant 100 mm spacing aimed at determining the distribution of shear forces. The concentrated loads (400 mm x 400 mm) are introduced by a 10 mm thick neoprene pad, on top of which there are four 200 mm x 200 mm x 40 mm steel plates centrally loaded by a single 40 mm thick steel plate. In between there are 30 mm diameter stainless steel spheres.

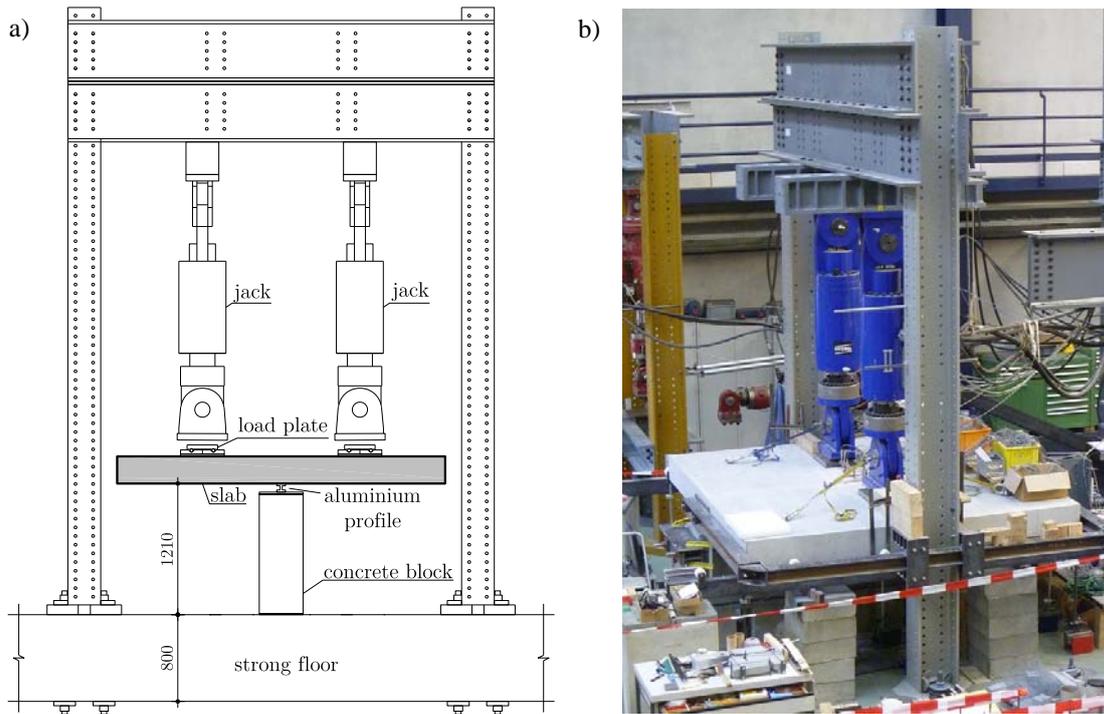


Fig. 3 Test setup: (a) elevation (dimensions in [mm]); (b) picture

2.2 Test specimens

The geometry and reinforcement layout of the tested slabs are presented in Fig. 4.

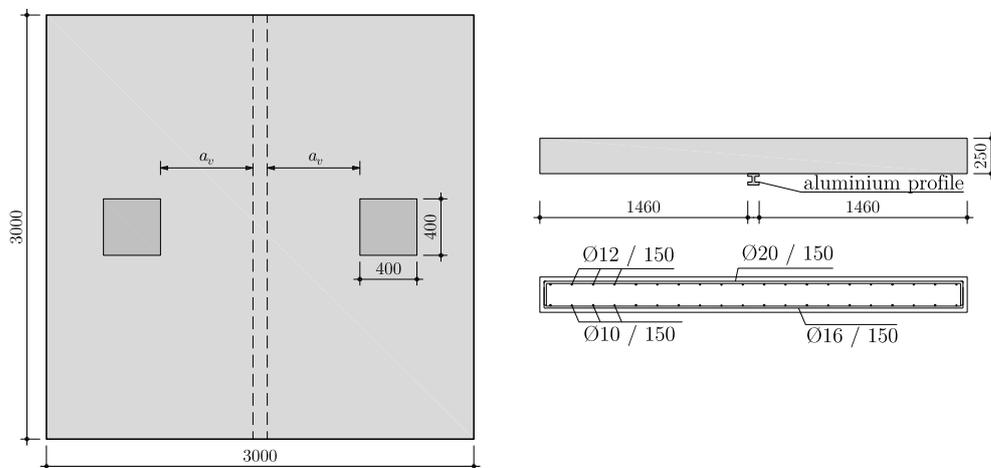


Fig. 4 Geometry and reinforcement layout of tested slabs (dimensions in [mm])

Normal strength concrete was used in all slabs, with compressive strengths f_c (measured on concrete cylinders, 320 mm high, 160 mm diameter) ranging from 32 to 47 MPa. The maximum aggregate size was 16 mm and the nominal concrete cover was 30 mm. The reinforcement consisted of ordinary deformed rebars with characteristic yielding stress of 500 MPa. The transversal nominal top reinforcement ratio was 1.0%

Two loading positions were adopted, corresponding to free shear spans a_v (refer to Fig. 4) of 440 mm and 680 mm, which correspond to 2.1 and 3.2 times the main nominal effective depth (d).

2.2 Test procedure

Two slabs were tested statically in order to obtain the reference static strengths (F_{Ref}) for each position of the load. Each slab provided two static tests. After the first shear failure on one side, the slab was strengthened with steel profiles on top and bottom faces connected with prestressing steel bars that passed through holes drilled on the slab after failure. The test was then continued until failure in the other side occurred.

The fatigue loading was done in a combined force-displacement control mode. The forces of the two actuators were not independently controlled. Instead, the average force of both jacks was controlled and the relative displacement between them was set to never be higher than 10 mm. Differences between maximum applied forces on both sides were smaller than 1% for five tested slabs, between 2-3% for other two slabs, and 3.1% for the remaining one.

The target ratio R between the minimum (F_{min}) and the maximum (F_{max}) applied forces was 0.10, and the actual values have varied between 0.09 - 0.12.

For each load position four different levels (LL) of maximum applied load were used. The maximum applied load was determined as follows:

$$F_{max} = LL f_c^{1/2} [F_{Ref} / f_{c,Ref}^{1/2}]_{avg}, \quad (1)$$

where f_c is the concrete compressive strength at the day of test start and $[F_{Ref} / f_{c,Ref}^{1/2}]_{avg}$ is the average static strength of the two reference tests normalized with the square root of the concrete compressive strength $f_{c,Ref}$ at testing day. For the free shear span $a_v = 680$ mm the target loading levels LL were 60, 70, 80 and 90%, and for $a_v = 440$ mm 80, 85, 90 and 95%.

When a slab failed in shear-fatigue on one side but not on the other, the slab was once again strengthened as previously explained. Table 2 resumes the main properties of the tested specimens.

Table 2
Main properties of tested specimens

Specimen	a_v , [mm]	type	the slab was strengthened?	target LL [%]
FN1	440	static	Y	-
FN2	440	fatigue	Y	95
FN3	440	fatigue	N	90
FN4	440	fatigue	Y	85
FN5	440	fatigue	N	80
FN6	680	static	Y	-
FN7	680	fatigue	N	90
FN8	680	fatigue	Y	80
FN9	680	fatigue	N	70
FN10	680	fatigue	N	60

3 Test results

3.1 Static reference tests

The quasi-statically tested slabs (reference specimens) failed in shear, in a similar manner as the tests reported in Natário, Fernández Ruiz & Muttoni (2014). For both loading positions, once the maximum load was attained the slabs presented a softening behaviour, with a significant decrease of the applied load for increasing displacements. The cracking pattern on the top surface developed parallel to the linear support in the central region, while on the bottom surface cracking was mostly perpendicular to the line support and concentrated in the loading area. The critical shear cracks were similar to typical shear cracks of beams without shear reinforcement. Their analysis shows that the cracks developed from a flexural crack at a given distance from the loading plate (and not from the tip of the loading plate as in typical punching shear failures). These facts justify the observed failures to be classified as shear failures.

3.1 Fatigue tests

All slabs but FN5 ($a_v=440$ mm; $LL=80\%$), FN9 ($a_v=680$ mm; $LL=70\%$) and FN10 ($a_v=680$ mm; $LL=60\%$) failed in shear-fatigue without rebar fractures. Fig. 5 presents the Wöhler diagrams for each loading position normalized by the average failure loads of the static reference tests. The maximum applied loads and the static shear strengths are on their turn normalized with the square-root of the concrete compressive strength. The determination of the cycle when the first 20 mm rebar failure took place was done through cross-interpretation of the strain evolution measured in strain gauges placed at the centre of some selected 20 mm rebars and the evolution of crack openings (devices to track crack opening evolution were placed at selected cracks after the first loading cycle).

Tests that failed in shear-fatigue presented similar cracking patterns as the static reference specimens, and the slabs which exhibited rebar fractures eventually failed in shear as well, due to excessive flexural crack openings that propagated into critical shear cracks.

Tests with free shear span $a_v = 680$ mm presented several transversal 20 mm rebar fractures located at the top surface in the centreline, as well as some 10 mm longitudinal rebar fractures on the bottom surface. These 10 mm bars are located at the transversal section that passes through the middle of the loading plates, between the load and the free edge. The determination of the cycle when these 10 mm bars failed was not possible.

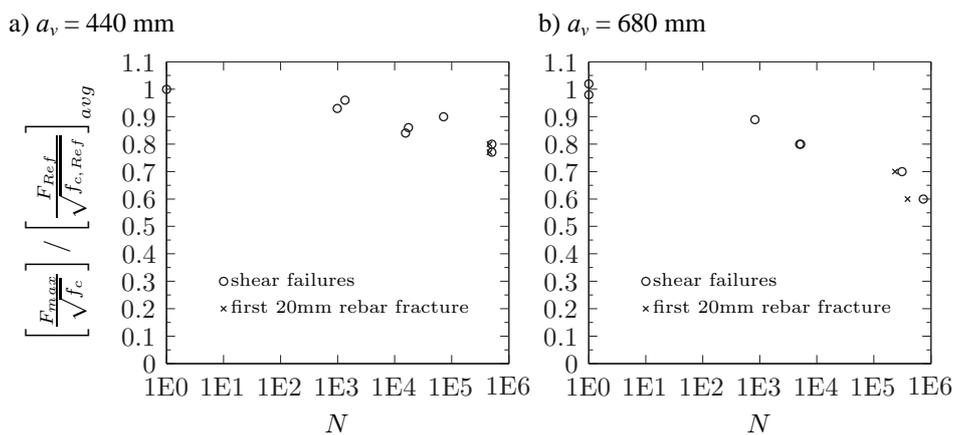


Fig. 5 Wöhler diagrams of tested slabs: (a) $a_v = 440$ mm; (b) $a_v = 680$ mm

The test with free shear span $a_v = 440$ mm which presented rebar fractures is somewhat different from previous cases. Three 20 mm rebars failed between the centreline and one loading plate, at the intersection between the critical shear crack that developed from a flexural crack and the main flexural reinforcement. Dowel action might have generated additional stresses in the rebars due to local bending. This might have potentially contributed to an increase of the fatigue damage in these bars.

All slabs that failed due to rebar fractures presented a significant remaining life after the first 20 mm rebar failure occurred. All bars failing under fatigue cycles were extracted from the tested specimens to observe the failure surface (Fig. 6).

4 Comparison between tests and *fib*-Model Code 2010 shear-fatigue provisions

4.1 Test on beams

Fig. 7 presents the comparison between the shear-fatigue provisions of the *fib*-Model Code 2010 (*fib* 2012) and tests on beams without shear reinforcement that failed in shear-fatigue. Only tests with a distance between the centre of the support and the centre of the load greater than three times the effective flexural depth are presented, to avoid any potential arching action. These criteria lead to a reduction from 100 to 87 tests of the comparison with the *fib*-Model Code 2010 published by Gallego, Zanuy & Albajar (2014).

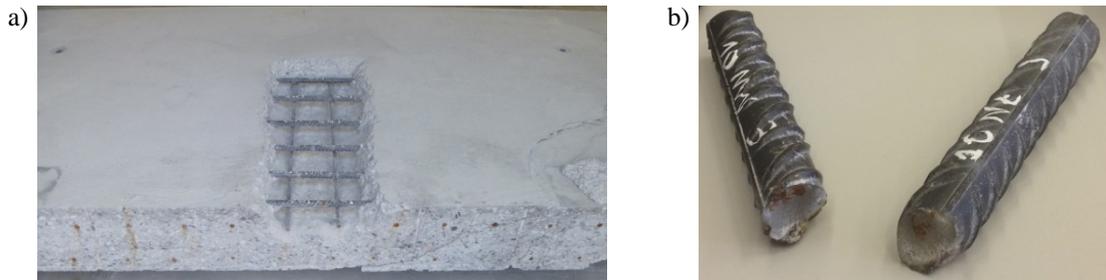


Fig. 6 Failed rebars: (a) concrete cover removed to access failed rebars; (b) extracted rebar and fatigue damage at the cross section

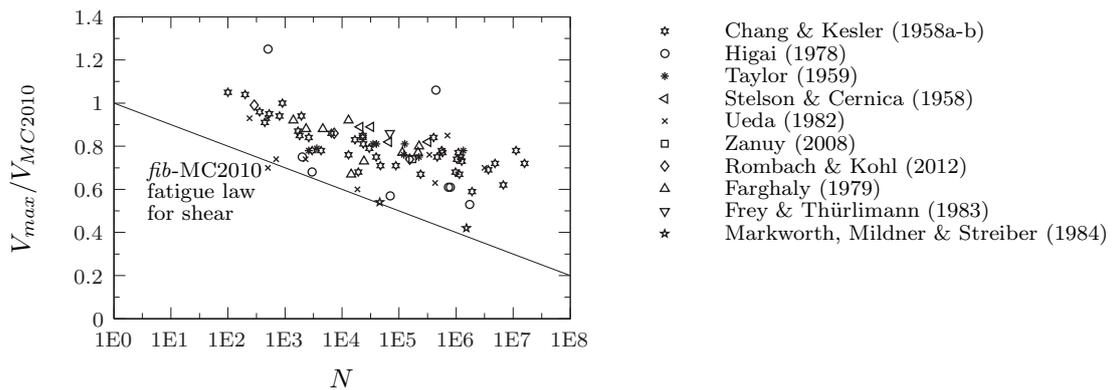


Fig. 7 Comparison between the *fib*-Model Code 2010 shear-fatigue provisions and the Wöhler diagram of tests on beams without shear reinforcement that failed in shear-fatigue (Gallego, Zanuy & Albajar 2014)

4.2 Tested slabs

Fig. 8 top presents the comparison between the shear-fatigue provisions of the *fib*-Model Code 2010 and the tested specimens. The static shear strengths required for this comparison were obtained with the effective width proposed by the code combined with the maximum acting bending moment at the control section, determined on the basis of a finite element model, assuming linear elastic behaviour of the slab (with the aluminium support behaving as a compression-only support). In Fig 8 bottom, the results are normalized to the measured strength of the slab under a quasi-static application of the load.

4.3 Discussion

Shear-fatigue failures on beams without shear reinforcement only seem to occur at maximum applied loads greater than 40% of the static shear strength calculated according to the *fib*-Model Code 2010, based on the Simplified Modified Compression Field Theory (Bentz, Vecchio & Collins 2006). The proposed shear-fatigue provisions are on the safe-side relative to experimental evidence (refer to Fig. 7).

The tests presented in this paper seem to indicate that typical European bridge deck slabs are less sensitive to shear-fatigue than beams without shear reinforcement. This aspect might be due to the

two-way behaviour that slabs under concentrated loads near linear supports exhibit, allowing redistribution of internal forces after local failures associated to fatigue damage. The *fib*-Model Code 2010 shear-fatigue provisions are again on the safe side when compared to the tests (Fig. 8 top). This holds true even if the code would be able to accurately predict the static shear strength of each slab (refer to Fig. 8 bottom). The fact that the code is not able to accurately predict the static shear strength for ratios $a_v / d < 3$ has been previously discussed elsewhere (Natário, Fernández Ruiz & Muttoni 2014).

These tests were performed under a fixed pulsating load. However the fatigue behaviour of a reinforced concrete slab can be deeply affected by moving loads (Perdikaris & Beim 1988; Perdikaris, Beim & Bousias 1989; Hwang & al. 2010).

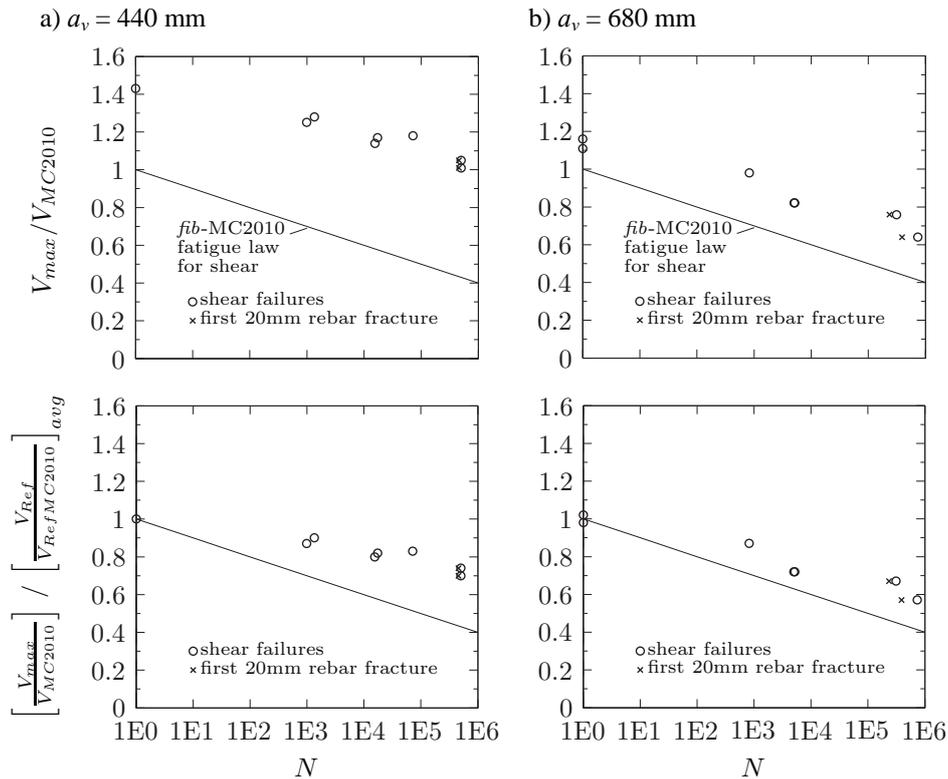


Fig. 8 Comparison between the *fib*-Model Code 2010 shear-fatigue provisions and the tested specimens: (a) $a_v = 440$ mm; (b) $a_v = 680$ mm

5 Conclusions

This paper presents an experimental campaign on the fatigue behaviour of reinforced concrete cantilever slabs subjected to concentrated loads near linear supports. The main conclusions of this paper are:

- Shear-fatigue failures on beams without shear reinforcement only seem to occur when the maximum applied load is greater than 40% of the static shear strength calculated according to the *fib*-Model Code 2010.
- Typical reinforced concrete cantilever bridges under concentrated loads seem to be less sensitive than beams without shear reinforcement to shear-fatigue phenomena.
- The *fib*-Model Code 2010 seems to underestimate the arching action effect for cantilever slabs subjected to concentrated loads.
- The *fib*-Model Code 2010 shear-fatigue provisions seem to be reasonably appropriate if an accurate prediction of the static shear strength is performed.
- Slabs that fail due to rebar fractures presented a significant remaining life after the first principal rebar failure occurred.

Acknowledgements

The authors would like to gratefully acknowledge the support and funding of the Swiss Federal Road Authority and Dr. Juan Manuel Gallego for the kind permission to use his shear-fatigue database on beams without shear reinforcement.

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