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SHEAR RESISTANCE OF FLAT SLAB BRIDGES AFFECTED BY ASR

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ABSTRACT

This paper deals with a study of the shear capacity of flat slab bridges, in which the tensile strength had been reduced due to alkali–silica reaction (ASR). Since the bridge decks are not provided with vertical reinforcement, their shear capacity fully depends on the concrete tensile strength. To study the effect of ASR on the shear capacity, six beams sawn from two viaducts were loaded in bending till failure. The shear span was varied between 2.5 and 4.5 times the beam depth. Both ends of each beam were subsequently tested.

Shear failure occurred at about 75% of the theoretical load without ASR. Contrary to what normally would be expected, failure was not attended with the development of inclined bending cracks, but with diagonal cracks that originated at mid-depth. Hence, the tensile strength reduction due to ASR resulted in a change of the failure mechanism from flexural shear into shear tension.

To explain the observed crack development and shear strength, the influences of a longitudinal compressive stress due to the restraint of ASR induced expansion and an orientation dependent tensile strength were considered. The latter was based on the measured uniaxial tensile strength that was lower in vertical than in horizontal direction.

Keywords: ASR, bridges, shear capacity, tensile strength

INTRODUCTION

Since the beginning of the nineties numerous viaducts in the Netherlands have been traced that suffer from ASR. Uniaxial tensile tests on cores drilled from the bridge decks sometimes showed a dramatic reduction of the tensile strength. Since these decks are not provided with shear reinforcement, the question was raised whether the remaining shear strength would still satisfy the requirements, the more so as in large parts of the decks mainly horizontally orientated cracks had been developed.

By order of Bouwdienst Rijkswaterstaat (Dutch Highway Authority) six beams, sawn from two 35 years old viaducts, were subjected to shear tests in the Stevin Laboratory at Delft University of Technology. The study is done in cooperation with the CUR Committee C106 'Structural consequences of ASR' (Centre for Research and Recommendation in Civil Engineering).

EXPERIMENTS

Specimens

The first two beams, indicated as ZB1 and ZB2, were sawn from the northern span of a three-span continuous slab bridge. The next four beams, indicated as HS1 to HS4, came out the southern span of a similar viaduct. In both cases this was the most affected area of the deck as was found from tensile tests on drilled cores. The length was 8.5 and 7.5 m for the ZB and HS beams, respectively. The depth of the ZB-beams increased from 0.65 m at the north end to 0.75 m at the south end and in case of the HS-beams from 0.6 m at the south end to 0.75 m at the north end. Other dimensions are given in Table 1.

Reinforcement - The reinforcement consisted of plain bars with 220 MPa nominal yield strength. Longitudinal and transverse bars were placed at the top and the bottom of the decks and only in the transverse beams over the supports and in the longitudinal edge beams vertical bars had been applied. The reinforcement ratios are given in Table 1.

Additional Reinforcement - Although in the ZB-tests diagonal shear cracks occurred, the beams did not fail in shear but in bending due to yielding of the steel. Moreover, the reinforcement ratio in the HS-beams was lower than in the ZB-beams and, finally, the ZB-beams had suffered more from ASR than the HS-beams. Therefore the HS-beams were strengthened by means of steel strips glued to the bottom side over the entire beam length, by which the flexural resistance is increased more than the shear resistance.

Concrete Properties - When in a structure ASR is suspected a basic research is carried out, which includes the measurement of concrete strength and stiffness on vertically drilled 75 mm diameter cylinders. On the basis of the results a more extensive investigation can be decided for. In case of viaduct HS this resulted in the strength values given in Table 2. The beams were sawn in the span that was limited by the end support and intermediate support at the south side.

The southern span close to the intermediate support had clearly suffered most from ASR. More striking, however, is the difference between the splitting tensile strength and the uniaxial tensile strength, for which with a 50 MPa cube strength would normally be expected 3.5 and 3.1 MPa, respectively. With respect to these values the splitting tensile strength is reduced by 10 to 20%, whereas the uniaxial tensile strength is reduced by about 70%. This difference can be understood by assuming that ASR damage results in disk like weakened spots. Due to the absence of vertical reinforcement these 'crack planes' will be mainly horizontally orientated, and a vertical cylinder may intersect some of these weakened planes; see Fig. 1. Thus the tensile strength is not uniformly distributed over the height of the cylinder. This results in the observed difference between the uniaxial tensile strength, which reflects the strength of the weakest section, and the splitting tensile strength, which gives the strength at the section where the splitting force is applied. The average splitting strength perpendicular to the axis of the vertically drilled cylinders is somewhat lower than parallel to the axis, which is in accordance with the visual observation that the ASR crack planes were mainly horizontally orientated. Larive (1998) observed anisotropy also in unreinforced specimens. Largest swelling and lowest tensile strength coincided with the casting direction.

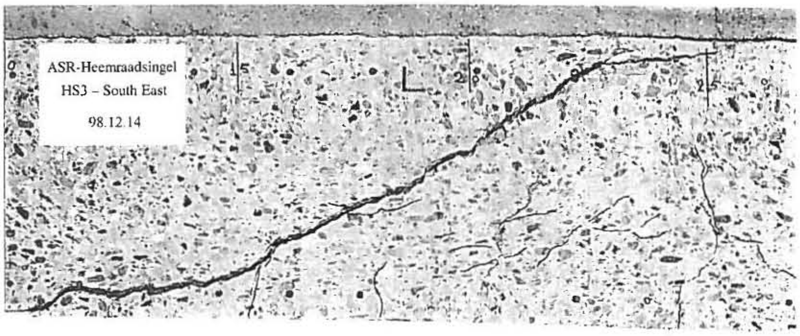

Fig. 1: Beam end HS3-South after shear failure. In this region of the beam some ASR damage was observed and marked in the lower part at the right side.

TABLE 1: Cross Sectional Dimensions and Reinforcement Ratio of Tested Beams

Test	b [mm]	d[1)] [mm]	Internal Reinforcement[2)]	External Reinforcement[3)]	$\omega_{0,int}$ [4)] [%]	$\omega_{0,ext}$ [4)] [%]	$\omega_{0,tot}$ [4)] [%]
ZB1-North	600	620	8_k28	-	1.32	-	1.32
ZB1-South	600	720	8_k28	-	1.14	-	1.14
ZB2-North	600	640	7_k28	-	1.12	-	1.12
ZB2-South	600	740	7_k28	-	0.97	-	0.97
HS1-North	480	720	2_k25+2_k22	3×12×120	0.50	1.25	1.75
HS1-South	480	670	2_k25+2_k22	3×12×120	0.54	1.34	1.88
HS2-North	508	705	2_k25+2_k22	3×6×120	0.49	0.60	1.09
HS2-South	508	645	2_k25+2_k22	3×6×120	0.53	0.66	1.19
HS3-North	520	730	2_k25+2_k22	3×6×120	0.46	0.57	1.03
HS3-South	520	680	2_k25+2_k22	3×6×120	0.49	0.61	1.10
HS4-North	570	680	2_k25+2_k22	3×12×120	0.45	1.12	1.57
HS4-South	570	670	2_k25+2_k22	3×12×120	0.46	1.13	1.59

1) Depth of section where inclined crack originated
2) FeB 220, yield stress 220 MPa, diameter in mm
3) Yield stress 410 MPa
4) Reinforcement ratio related to dimensions of cross-section where inclined crack originated

TABLE 2: Compressive and Tensile Strength in Bridge Deck HS Measured on 24 Vertically Drilled Cylinders 75 mm (Visser and Siemes 1998)

Type of Test	End Support South	Intermediate Supp. South	Intermediate Supp. North	End Support North	Average
Cube compressive	50.6 / 4[1)]	50.4 / 24	54.0 / 10	60.6 / 9	53.9 / 14
Splitting tensile ⊥[2)]	3.53 / 22	2.85 / 13	3.19 / 10	3.82 / 10	3.35 / 17
Splitting tensile //[3)]	3.45 / 11	3.16 / 5	3.37 / 14	4.21 / 14	3.55 / 16
Uniaxial tensile	1.02 / 48	0.84 / 45	0.89 / 26	1.61 / 17	1.11 / 42

1) i/j means average is i MPa and COV (coefficient of variation) is j %

2) ⊥ means splitting plane perpendicular to core axis

3) // means splitting plane parallel to core axis

To quantify the anisotropy the tensile strength was measured in the principal directions of the ZB-viaduct (Visser et al. 1998). In the northern span, from which the beams were sawn, the average uniaxial tensile strength in vertical and in horizontal direction amounted to 0.6 MPa (COV 26%, 15 tests) and 1.20 MPa (COV 28%, 14 tests), respectively.

The influence of the load direction on the splitting tensile strength was much less than for the uniaxial tensile strength. The average of all splitting tensile tests was 2.83 MPa (COV 20%, 21 tests). The cube compressive strength was about 60 MPa, which means that the reduction of both the uniaxial tensile strength and the splitting tensile strength was relatively more than for the HS-viaduct (see also Table 2).

TABLE 3: Ultimate Shear Force from Tests and Theory

Test	$V_{u,test}$ [kN]	$\tau_{u,test}$ 1) [MPa]	a/d	φ [°]	$V_{u,theor}$ 2) [kN]	$\frac{V_{u,test}}{V_{u,theor}}$	Failure Mode 3)
ZB1-North	354	1.42	3.07	21	525	0.67	Bending/wide SC
ZB1-South	380	1.32	2.64	23	675	0.56	Bending/fine SC
ZB2-North	330	1.29	2.97	30	522	0.63	Bending/fine SC
ZB2-South	354	1.20	2.67	-	640	0.55	Bending/no SC
HS1-North	349	1.52	4.17	35	421	0.83	Shear
HS1-South	330	1.54	4.48	32	402	0.82	Shear
HS2-North	361	1.51	2.55	29	507	0.71	Shear
HS2-South	321	1.47	2.79	28	427	0.75	Shear
HS3-North	369	1.46	2.47	29	541	0.68	Shear
HS3-South	350	1.49	2.65	28	477	0.73	Shear
HS4-North	374	1.45	3.61	32	470	0.80	Shear
HS4-South	380	1.49	3.88	25	462	0.82	Shear

1) $\tau_{u,test} = 1.5\tau_{avg} = \frac{V_{u,test}}{bd}$ for a parabolic shear stress distribution over the beam depth

2) According to Eq. [1] (Rafla, 1971)

3) SC = Shear Crack

Execution and Results of the Experiments

Both sides of the each beam have subsequently been loaded until failure in a four-point-bending test, with non-symmetric positioning of the loads. The tested part of a beam was provided with steel clamps before the second test was done. The load was stepwise applied with two hand-operated jacks in about 2 hours. Loads, deflection and concrete strains along the shear span were automatically measured. At each load step crack development was marked.

The main results of the experiments are summarized in Table 3. All ZB-beams failed in bending, although in ZB1 wide shear cracks had already developed when the reinforcement started to yield. The reinforcement ratio in ZB2 was less (see Table 1), which explains why bending failure occurred at a lower load. All HS-beams failed in shear and no yielding occurred. The average value of the ultimate shear stress of the ZB (except ZB2-south) and HS-beams is 1.34 and 1.49 MPa, respectively. The difference between these two values corresponds to the ASR-attack rate as was visually observed and found in the aforementioned tensile tests. The variation of the shear span to depth ratio between 2.5 and 4.5 did not influence the shear strength.

ANALYSIS OF THE SHEAR RESISTANCE

Mode of Failure and Shear Strength without ASR

In case of rectangular beams without shear reinforcement flexural shear is normally the shear failure mechanism. Bending cracks will incline and develop towards the loading point until the compression zone fails. In the present case, however, failure was caused by shear tension, as was reflected by the initiation of inclined cracks at the neutral axis of the beam. Without ASR the expected average load at shear failure is found according to Rafla (1971) from:

$$V_{u,theor} = \alpha_u d^{-0.25} \sqrt{f_{cc}} \sqrt[3]{\omega_0} bd \qquad [1]$$

with $\alpha_u = 0.90 - 0.03\dfrac{a}{d}$ for $\dfrac{a}{d} \geq 3.5$

$\alpha_u = 0.795 + 0.293\left(3.5 - \dfrac{a}{d}\right)^{2.5}$ for $2.0 \leq \dfrac{a}{d} \leq 3.5$

Substituting in Eq. [1] the corresponding values from Table 1 and Table 2 and $f_{cc} = 60$ and 55 MPa for ZB and HS, respectively, yield the theoretical ultimate shear loads given in Table 3. The average ratio between the experimental and the theoretical strength amounts to 0.60 (COV 9.5%) for the ZB-beams and to 0.77 (COV 7.5%) for the HS-beams. The difference between these two ratios can be ascribed to a combination of the facts that the ZB-beams did not fail in shear but in bending and that the ZB-beams suffered more from ASR than the HS-beams.

Influences of ASR on Shear Resistance

Two consequences of the occurrence of ASR may influence the structural behaviour under shear loading. First, compressive stresses may be developed in the concrete when the volume increase due to ASR is - partly - restrained by reinforcement. With such a restraint the reduction of the strength, especially the tensile strength, is less than without restraint. Second, when the restraint is not available in all principal directions the tensile strength will depend on the orientation.

By equating σ_φ and $f_{ct,\varphi}$, the ratio between the shear stress τ_{xy} and the tensile strength $f_{ct,90}$ is obtained as a function of the inclination φ, the tensile strength ratio β and the prestress ratio ρ (normal stress σ_x divided by tensile strength $f_{ct,90}$). The value of φ for which this expression has its minimum, gives the inclination of the failure plane. The latter is illustrated in Fig. 4, where the resulting inclination φ and normal stress $\sigma_\varphi/f_{ct,90}$ for $\beta=0.5$ and some values of ρ are given by the points of contact between the lines for the tensile strength and the normal stress on the failure plane. Substitution of these values in Eq. [5] yields the ultimate shear stress ratio $\tau_{xy}/f_{ct,90}$.

Discussion of Analysis Results

The inclination of the failure plane φ and the ultimate shear stress $\tau_{xy}/f_{ct,90}$ calculated with the aforementioned failure criterion have been displayed in Fig. 5 as a function of the tensile strength ratio β ($f_{ct,0}/f_{ct,90}$) and some values of the prestress ratio ρ.

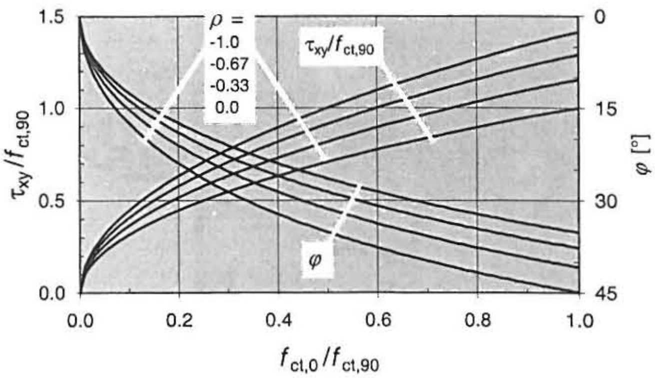

Fig. 5: Ultimate shear stress and inclination of failure plane as a function of tensile strength ratio and prestress ratio

Some results derived from Fig. 5 have been summarized in Table 4. For a comparison with the test results, the observed inclination of the shear cracks is taken as a starting point. In the experiments on the HS-beams an average inclination of 30° (COV 10%, 8 results) was found and the average ultimate shear stress was 1.5 MPa; see Table 3. Further criteria for the comparison are the vertical tensile strength and the prestress.

The average vertical uniaxial tensile strength in the region where the HS-beams were sawn was 0.93 MPa; see Table 2. This value agrees reasonably with the vertical strength found in the analysis, which appeared to depend only on the crack inclination. In horizontal direction the tensile strength had not been measured, but in the apparently more damaged ZB-viaduct the ratio between the vertical and horizontal tensile strength was 0.5. For such a value of the tensile strength ratio, the analysis yields a prestress ratio of –0.5 and, hence, a prestress of –0.9 MPa, which is clearly higher than the aforementioned average value of –0.3 MPa, derived from steel strain measurements on HS-beams. With the latter prestress, which would mean a prestress ratio of –0.17, the analysis gives a tensile strength ratio of about 0.4, which contradicts the earlier assumption in this respect. Further investigation is required to determine whether this inconsistency is caused by a wrong estimation of e.g. the tensile strength ratio or the prestress or by shortcomings of the applied failure criterion.

TABLE 4: Summary of Analysis Results

φ [°]	$\rho^{2)}$	$\beta^{3)}$	$\tau_{xy}/f_{ct,90}$ 3)	$f_{ct,90}$ 4) [MPa]	$f_{ct,0}$ 5) [MPa]	σ_x 6) [MPa]
25	−0.33	0.28	0.60	2.50	0.70	−0.83
30 1)	−2.00	1.00	1.73	0.87	0.87	−1.73
30	0.00 −0.33 −0.67 −1.00	0.34 0.45 0.56 0.67	0.58 0.76 0.95 1.15	2.59 1.97 1.58 1.30	0.88 0.89 0.88 0.87	0.00 −0.65 −1.06 −1.30
35	−0.33	0.67	0.92	1.63	1.09	−0.54

1) This row follows from Fig. 2; 2) Assumed; 3) Read from Fig. 5;
4) $f_{ct,90} = \tau_{xy,test}/(\tau_{xy}/f_{ct,90})$; $\tau_{xy,test}$ = 1.5 MPa; 5) $f_{ct,0} = \beta \cdot f_{ct,90}$; 6) $\sigma_x = \rho \cdot f_{ct,90}$

An impression of the sensitivity of the calculated shear stress to the choice of various parameters can be found from Table 5. Assuming that the vertical tensile strength can fairly be measured, the prestress and the tensile strength ratio have been varied.

TABLE 5: Sensitivity of Calculated Shear Strength to Parameter Choice

$f_{ct,0}$ [MPa]	σ_x [MPa]	β	ρ	$\tau_{xy}/f_{ct,90}$	τ_{xy} [MPa]
0.9	−0.2	0.4 0.6	−0.09 −0.13	0.66 0.82	1.48 1.23
0.9	−0.4	0.4 0.6	−0.18 −0.27	0.70 0.86	1.57 1.29

The vertical tensile strength used in Table 5 corresponds to the average value measured in the HS-beams, where the average shear strength was found to be 1.5 MPa. It can be seen that the risk to overestimate the shear strength increases as the value for the prestress is put higher or the tensile strength ratio is put lower.

CONCLUSIONS

The shear capacity of flat slab bridges without shear reinforcement that suffered from ASR has been studied. Six beams sawn from two bridges were loaded in bending until failure and a theoretical analysis was carried out, in which the effect of an orientation dependent tensile strength on the shear capacity was investigated. The following conclusions were drawn.

1. Due to the reduction of the tensile strength caused by ASR, the failure mechanism is of the shear tension type, whereas without ASR damage flexural shear failure would have been expected.

2. The average capacity in case of shear failure was 75% of the value that would have been expected when no ASR damage had occurred.

3. The crack inclination of about 30° could not be explained with the help of an ASR induced prestress alone.

4. By assuming an orientation dependent tensile strength the experimental findings could be made plausible. This assumption involves a failure criterion for an anisotropic material that needs further verification.

5. The shear resistance of a member without shear reinforcement that suffered from ASR can be estimated on the basis of the tensile strength in vertical direction, the tensile strength ratio and the prestress caused by partly restraint ASR expansion.

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NOTATIONS

a	shear span
b	width of concrete section
d	depth of concrete section
f_{cc}	cube compressive strength
f_{ct}	concrete tensile strength
$f_{ct,\varphi}$	tensile strength on plane with inclination φ
$f_{ct,0}$	tensile strength for $\varphi=0°$
$f_{ct,90}$	tensile strength for $\varphi=90°$
$V_{u,test}$	ultimate shear force in test
$V_{u,theor}$	ultimate shear force in according to theory
COV	coefficient of variation
β	tensile strength ratio $= f_{ct,0}/ f_{ct,90}$
φ	inclination of plane with respect to longitudinal axis
ρ	prestress ratio $= \sigma_x / f_{ct,90}$
σ_x	normal stress in longitudinal direction (for prestress negative)
σ_φ	normal stress on φ-plane
τ_{xy}	shear stress
$\tau_{u,test}$	ultimate shear stress in test
τ_φ	shear stress on φ-plane
ω_0	reinforcement ratio in %