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Experiments on skewed reinforced concrete slabs failing in shear

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Abstract

Reinforced concrete solid slab bridges are often skewed to cross underlying objects, which increases the shear stress concentration at the obtuse corner. Limited experimental evidence on skewed slabs is available, so that both the shear capacity and failure mode in skewed slab bridges are subject to discussion. Therefore, an experimental program at Delft University of Technology investigated the capacity and failure modes in skewed slabs under concentrated loads near the edge. Results from 15 tests on five 1:2-scale slab members result in shear failures and show a decreasing capacity with increasing skew angles. The obtuse corner is found to be critical; the reinforcement layout did not influence the capacity significantly. Comparisons with calculation methods showed reasonable accuracy. A proposed method using a larger integration length around the peak shear stress obtained from linear finite element modeling may be recommended for assessment.

Keywords: assessment; experiments; finite element modelling; flexure; reinforcement layout; shear; skewness; slabs; slab bridges.

1 Introduction

Many existing reinforced concrete solid slab bridges are not straight but present a skew angle to cross the underlying object. An analysis of the existing reinforced concrete slab bridges in the Netherlands led to the conclusion that about half of all existing slab bridges in the Netherlands are skewed slabs. The skew angle is known to influence the flow of stresses, resulting in a larger concentration of shear stresses in the obtuse corner.

In terms of terminology, it should be remarked that in the Netherlands a straight slab is considered a 90° slab. As such, slabs are referred to by their crossing angle rather than their skewness. For consistency with national practice, we will use crossing angle in this paper as well.

At the same time, limited experimental evidence on skewed slabs is available, so that both the shear capacity and failure mode in skewed slab bridges is subject to discussion. The most insightful series of experiments carried out in the past on skewed slabs are the experiments done in Liverpool in the early 1980s [1, 2]. These slabs were small scale (scale 1:5 of a slab bridge), and loaded with the live load model from the British Standard governing at that time. The crossing angles that are used in these experiments are 60°, 45° and 30°. Analysing these experiments is not straightforward, as the failure mode changed between the different crossing angles: shear at 60° crossing angle, flexure-induced punching of three supports at 45°, and punching of a single support at 30°. As such, these experiments do not provide information on the reduction of the shear capacity as a function of the skew or crossing angle.

There is a need for experiments that are larger scale, designed to result in shear failures at all crossing angles, and with modern instrumentation to obtain additional insights.

2 Experiments

2.1 Experimental design

To address the aforementioned research need, an experimental program is conducted at Delft University of Technology that investigates the maximum load and failure mode on skewed reinforced concrete slabs tested under a concentrated load at the shear-critical location.

The slabs are 1:2-scale representations of reinforced concrete slab bridges. They are designed to fail in shear, and their design is developed after a survey of the properties of existing skewed slab bridges in the Netherlands, to ensure the practical value of the experiments [3]. Moreover, the design of the specimens is aligned with previous straight slabs tested at Delft University of Technology [4, 5], for comparison to the behaviour of straight slabs.

In total, 15 experiments are carried out on five skewed slab members. All slabs have a span length of 3.6 m, which resulted in the total length to be adjusted as a function of the crossing angle. All slabs are 2 m wide and 300 mm thick.

Two reinforcement layouts are used: an orthogonal and non-orthogonal layout. The orthogonal layout has the longitudinal and transverse reinforcement crossing at 90° whereas the non-orthogonal layout applies the longitudinal and transverse reinforcement in parallel to the free edges of the specimen. All reinforcement bars are ribbed B500B steel, and the longitudinal reinforcement percentage of all slabs is 0.996%. Concrete C35/45 is used for all slabs.

2.2 Test setup and measurements

All slabs are simply supported and loaded in a displacement-controlled manner at 0.04 mm/s using a hydraulic jack and loading plate of 200 mm × 200 mm. Figure 1 shows a photograph of the test setup.

The slabs are supported on an HEA 700 steel beam and two box girders of 800 mm high at both ends. To create the support condition, a large steel plate with welded cylinder was bolted to the HEA 700 beam. Fixing the bolts results in a pin condition and loosening the bolts in a roller condition. Additionally, two steel plates of 100 mm wide with a layer of Teflon in between are used to allow horizontal displacements between top and bottom plates.

The instrumentation on the specimens consists of: LVDTs (linear variable differential transformers) to measure displacements that can be translated into strains, laser sensors to measure deflections, acoustic emission sensors and smart aggregates to analyse the internal cracking, and both 2D and 3D DIC (digital image correlation) to follow cracking during the test, to derive strain values, and to estimate the internal crack opening by comparing displacements top and bottom of the slab.

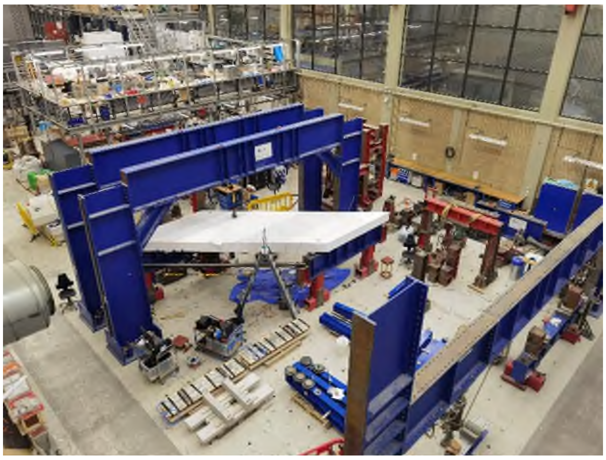

Figure 1. Skewed slab in test setup.

3 Results

3.1 Test results and failure mode

Table 1 gives an overview of the test results of the experiments. The experiments use the following format for the naming: S60N2Ac1. The latter example stands for: S for slab experiments, 60 for the crossing angle, N for non-orthogonal reinforcement layout (as compared to O for orthogonal) followed by the specimen number "2" in this case, A for the position of the load (A with a/d_l = 3 near the obtuse corner, B with a/d_l = 4 near the obtuse corner, C with a/d_l = 3 near the acute corner, D with a/d_l = 4 near the acute corner, where a is the shear span taken as the centre-to-centre distance between the load and the support, and d_l is the effective depth to the longitudinal reinforcement), c for testing on a previously cracked member (as opposed to uc for the uncracked specimens), and finally the test number. In addition, Table 1 gives the age of the specimen at the time of testing, and the average cube compressive strength obtained at this age $f_{cm,cube}$ as well as the maximum externally applied load in the experiments, P_{max}.

*Table 1. Overview of test results. Tests indicated with * are carried out near the acute corner.*

Test	Age [days]	$f_{cm,cube}$ [MPa]	a/d_l	P_{max} [kN]
S60N2Ac1	102	64.7	3	615.3
S60N2Ac2	112	64.7	3	573.8
S60N2Cc	113	64.7	3*	641.2
S60O1Auc	123	69.4	3	580.1
S60O1Ac	131	69.4	3	544.5
S60O1Cc	132	69.4	3*	673.9
S60N1Auc	152	65.1	3	606.4
S60N1Bc	162	65.1	4	532.6
S60N1Dc	163	65.1	4*	636.8
S45N1Auc	259	72.3	3	420.8
S45N1Ac	274	72.3	3	508.4
S45N1Cc	274	72.3	3*	572.1
S75N1Auc	302	70.47	3	745.9
S75N1Ac	308	70.47	3	599.7
S75N1Cc	308	70.47	3*	672.5

All 15 experiments resulted in a shear failure visible at the edge of the specimen, and with indications of breaking off of the obtuse corner, when testing near the obtuse corner. Figure 2 shows the DIC-processed images of cracking of a 60 degree specimen, indicating the shear crack on the side face and bottom face of the specimen.

3.2 Influence of parameters

The parameters tested in these experiments are: the effect of skewness, the reinforcement layout, and the difference between testing at the obtuse and acute corner. The effect of skewness is expected to reduce the shear capacity as skewness increases as a result of the shear stress concentrations in the obtuse corner. Figure 3 shows the test results, confirming indeed that the maximum load decreases almost linearly as the skewness increases.

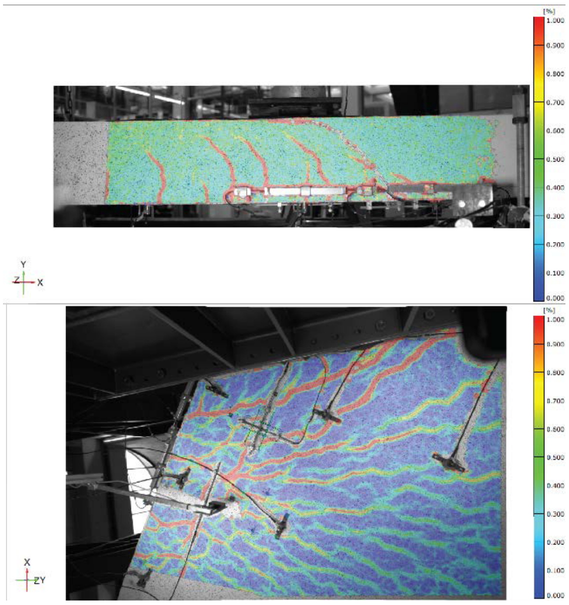

Figure 2. Shear cracking of S60N2Ac2.

In terms of reinforcement layout, it is expected that the layout that most aligns with the direction of the principal bending moments will be the most efficient, and those have the larger capacity as these bars cross the cracking plane perpendicularly. In non-orthogonal layouts, the transverse bars are parallel to the cracking plane, so only the projection of the longitudinal bars can be taken into account. Therefore, it is expected that the orthogonal layout will result in a larger capacity than the non-orthogonal layout.

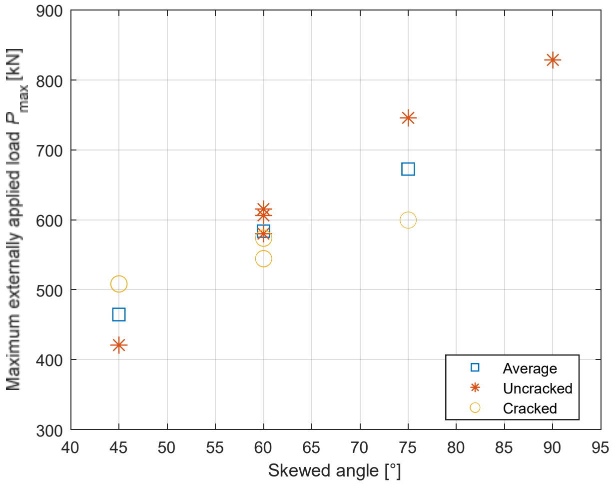

Figure 3. Relation between skew angle and maximum load at failure.

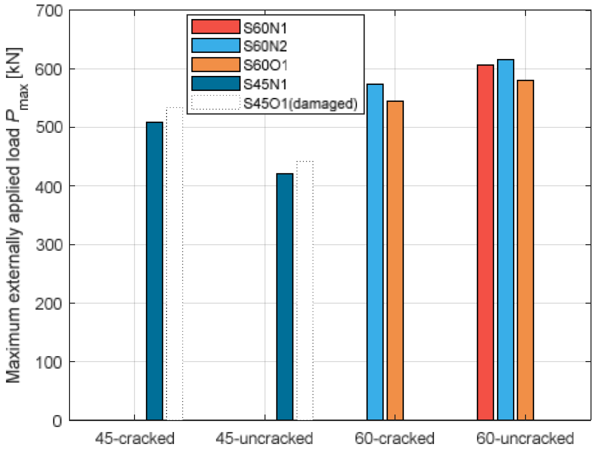

Figure 4. Effect of reinforcement layout.

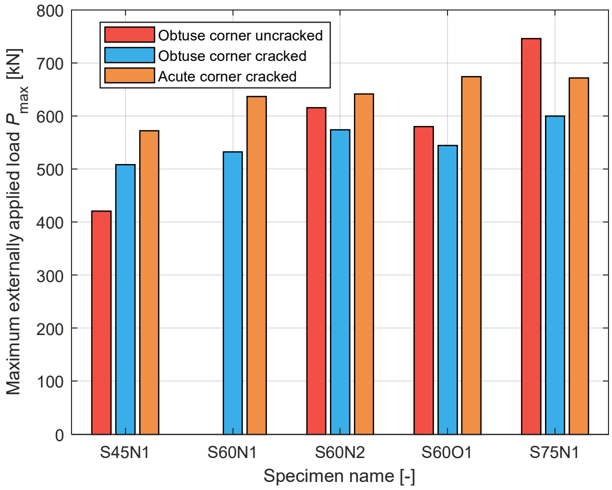

Figure 5. Testing at obtuse versus acute corner.

Figure 4 shows the experimental results. While the differences between the members with orthogonal and non-orthogonal reinforcement layouts are small, the results do show consistently that the members with non-orthogonal reinforcement result in a larger maximum load at failure. One potential explanation is related to the influence of the shear cracking over the flexural cracking, and the rotation of the cracking plane. Nevertheless, further research is needed to address the influence of the reinforcement layout.

Finally, from structural mechanics, it is expected that the obtuse corner is governing over the acute corner in terms of shear capacity, as the shear stress concentrations are largest in the obtuse corner. Figure 5 shows the difference between testing at the obtuse and acute corner, confirming the hypothesis that the obtuse corner is governing, when consistently comparing previously cracked members.

3.3 Comparison to calculation methods

To compare the test results to calculation methods, the emphasis is placed on methods that are used for the assessment of existing reinforced concrete slab bridges: the upcoming Eurocode [6], and the previous (RBK 1.1 [7]) and current (RBK 1.2 [8]) versions of the RBK (Guidelines for the assessment of bridges, prescribed in the Netherland for the assessment of highway bridges owned by Rijkswaterstaat).

For all three approaches, two Levels of Assessment [9] are applied: LoA I uses a hand calculation and effective width assumption for distribution of the shear, and LoA II is based on a linear finite element model and distribution width for the integration of the shear stresses around the peak value [10]. All comparisons are based on average values, without partial factors, to have a direct comparison with the experimental results. For all comparisons, both the shear and moment capacity are calculated, and the governing of the two is used as the prediction of the capacity.

Table 2 gives the comparison between the test results and six evaluation methods. Comparing the test results to calculation methods shows reasonable results with calculation methods. In particular, it should be noted that the maximum coefficient of variation is 14.3%, which is an excellent result for the complex problem of shear in slabs with the effect of skewness. This result is thanks to the improvements that have been made in recent years in the shear assessment of slab bridges and slabs, based on previous shear testing of slabs at Delft University of Technology. The lowest scatter is obtained using RBK 1.1 LoA II, and the average value closes to one is obtained using RBK 1.2 LoA I. The LoA I approach with the upcoming Eurocode results in overly conservative estimates of the capacity.

The Levels of Approximation method is not confirmed when using the current Dutch guidelines for the assessment of existing bridges RBK. The LoA approach is based on the expectation that increasing Levels of Approximation require increasing amounts of time and computational effort, but will lead to improved estimates of the capacity. For the current approaches, the hand calculations result in averages closer to one. The increased LoA does result in a reduction of the coefficient of variation.

Table 2. Comparison between test results and calculation methods, showing statistics of tested/predicted values of the capacity

Method	**Average [-]**	**Standard deviation [-]**	**Coefficient of variation [%]**
New Eurocode—LoA I	2.13	0.26	12.02
New Eurocode—LoA II	1.14	0.12	10.34
RBK1.1—LoA I	1.11	0.14	12.60
RBK1.2—LoA I	1.08	0.15	14.30
RBK1.1—LoA II	1.50	0.14	9.20
RBK1.2—LoA II	1.46	0.20	13.40

4 Proposed method

4.1 Description

A proposed method using a larger shear stress integration around the peak value together with a linear finite element model is expected to improve the assessment strategies for existing skewed reinforced concrete slab bridges. In this method, a distance of $8d_l$ around the load is used. The capacity $v_{Rm,c}$ is determined using the following method proposed by the current Eurocode [11]:

$$v_{Rm,c} = C_{Rm,c} k_h (100 \rho_l f_c)^{1/3} \tag{1}$$

with ρ_l the longitudinal reinforcement ratio, f_c the average value of the cylinder compressive strength of the concrete in MPa, and k_h the size effect factor:

$$k_h = 1 + \sqrt{\frac{200}{d_l}} \leq 2 \tag{2}$$

with d_l the effective depth to the longitudinal reinforcement in mm. The value of $C_{Rm,c}$ is equal to $0.18/\gamma_c$. The design calibration factor is equal to 0.12. The average calibration [12, 13] factor is assumed to be equal to 0.15 [14]. The reinforcement ratio ρ_l is determined for skewed members using ρ_{eff}, which for orthogonal layouts is:

$$\rho_{\text{eff},\alpha} = \rho_\text{l} \sin^2 \alpha + \rho_\text{t} \cos^2 \alpha \tag{3}$$

and for non-orthogonal layouts:

$$\rho_{\text{eff},\alpha} = \rho_\text{l} \sin^2 \alpha \tag{4}$$

According to Model Code 2010 [15], it is decided to use the section $0.5d_l$ away from the support as the control section. Using a linear finite element model, the average shear stress $\tau_{\text{avg,8d}}$ within $8d_l$ is then determined. It shall be mentioned that the total internal shear stress v_{total} is used, which is defined as the geometric average of the shear stress in the x- (v_x) and y-directions (v_y):

$$v_\text{total} = \sqrt{v_x^2 + v_y^2} \tag{5}$$

By utilizing the Eqs. (3) and (4), the influence of the twisting moments can be taken into account to some extent since the angle α should align with the maximum principal moment direction. However, in the current method, the angle α is assumed to be the same as the skewed angle based on the crack pattern observed in experiments.

On the other hand, when calculating the internal shear force, the shear force projected at the maximum principal moment direction should be considered for demand, while the total shear force is used instead in the proposed method. This can be considered as a conservative approach to incorporate the influence of the twisting moments.

4.2 Validation with test results

The proposed method is compared with the test results in Figure 6, and the relation to the skew angle and the tested/predicted results is given in Figure 7. It can be seen that the proposed 8d_l method predicts the shear capacity reasonably well regardless of the skew angle of the specimen. The influence of the skewness appears to be properly considered by using a linear finite element analysis and integrating the shear stresses over a distance of 8d_l around the peak value, at a section of 0.5d_l away from the support.

The average value of the tested to predicted shear capacity is 0.97 with a coefficient of variation of 10.3%. As such, this method has results of the average tested/predicted values closest to 1 (compared to the methods in Table 2), which indicates that this method is suitable for the assessment of existing concrete slab bridges.

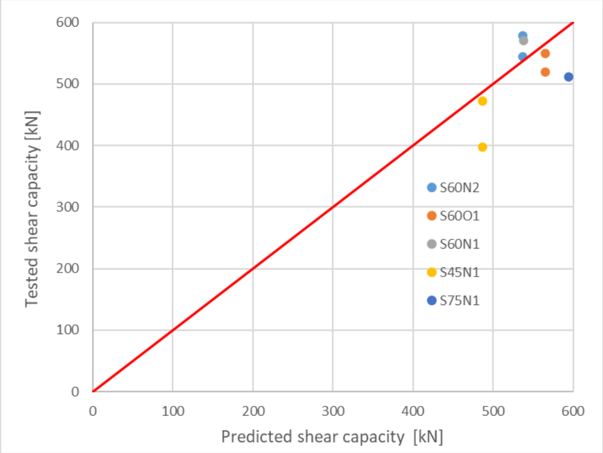

Figure 6. Comparison between proposed method and test results.

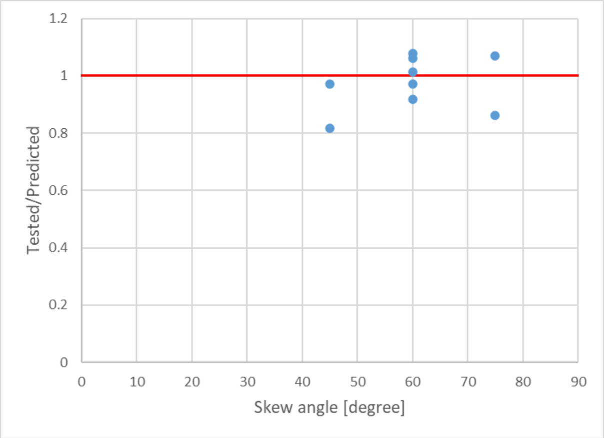

Figure 7. Dependency of proposed method on skew angle.

Moreover, the proposed method reaches a similar level of accuracy as the new Eurocode with LoA II, without the need for determining the effective shear span as required in the upcoming Eurocode. In other words, our proposed method is more straightforward to apply, and may thus be more suitable for assessment. However, more research is still needed to explicitly consider the influence of the twisting moments in the calculations of internal force and resistance.

5 Discussion

The presented experiments are, to our knowledge, the first comprehensive series of experiments on skewed slabs under concentrated loads that consistently result in shear failures. As such, these experiments are suitable for shining a new light on the discussion of the shear capacity of skewed members. This topic is very relevant for the assessment of existing concrete slab bridges, as about half of these bridges are executed under a skew angle to facilitate crossing the underlying object.

In terms of the effect of different parameters tested in these experiments, the topic of the reinforcement layout requires further study. Ideally, additional experiments need to be conducted. Moreover, bridge testing can be recommended to further study this effect. Finally, and more practically, nonlinear finite element models can be used to study the effect of the reinforcement layout in more detail and these can be calibrated with the available experiments before carrying out parameter studies.

An observation from the analysis of the predicted results and the resulting statistics of the tested/predicted ratios (see Table 2) shows low values of the coefficient of variation for the evaluated methods. For the problem under study, which is a combination of shear, slab behaviour, with the new complexity of the skew angle, this results is remarkable. However, this result reflects the benefits gained from over fifteen years of testing reinforced concrete slabs at Delft University of Technology, which has resulted in practical ways of determining the shear capacity of slabs (including the use of an effective width for shear, and the recommended methods for integrating the shear stresses) that lead to improved assessments and results when compared to experiments.

6 Summary and conclusions

This paper reports on 15 shear experiments carried out on five reinforced concrete solid slabs, representative of reinforced concrete solid slab bridges, failing in shear. These experiments resulted in the following conclusions:

- The specimens are designed to lead to shear failures. The specimens behaved as designed, and all experiments are shear failures.
- The shear capacity at the obtuse corner is lower than the shear capacity at the acute corner.
- The load at failure decreases almost linearly as the skewness increases (or as the crossing angle decreases).
- The influence of the reinforcement layout requires further study.
- All studied methods to predict the capacity lead to good results, with a maximum coefficient of variation of 14.3%.
- Of the studied methods, the RBK 1.1 Level of Assessment II method (using a linear finite element model) results in the lowest value of the coefficient of variation.
- Of the studied methods, the RBK 1.2 using Level of Assessment I (using a hand calculation and effective width) the average value of tested/predicted capacity closest to one.
- The proposed method of taking the shear stress at $0.5d_l$ from the support and integrating the shear stresses within $8d_l$ around the peak shear stress leads to good results, and can be recommended for the assessment of skewed reinforced concrete slab bridges.

In conclusion, the presented experiments shine a new light on the shear capacity of skewed reinforced concrete slab bridges, and can form the basis for further (numerical) research as well as contribute to the assessment of existing reinforced concrete slab bridges.

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