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PROLONGING THE SERVICE LIFE OF EXISTING REINFORCED CONCRETE SLAB BRIDGES THROUH EXPERIMENTAL STUDIES ON THE SHEAR CAPACITY

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ABSTRACT:
A large number of existing reinforced concrete slab bridges are found to be insufficient for shear when calculated according to the governing codes. Seeking improved methods, for example, based on new experimental evidence, to assess the residual shear capacity and prolonging their service life can avoid large economic, environmental and social costs. Experimental results are combined with Monte Carlo simulations to quantify the increase in shear capacity in slabs as a result of transverse load redistribution. As a result, a larger number of slab bridges can remain in service.
Keywords: assessment practice, slab bridges, shear, experiments, statistical evaluation, effective width, code extension

1. INTRODUCTION

1.1 Existing Bridges in the Netherlands

A large number of the existing reinforced concrete bridges in the road network of the Netherlands consist of short span solid slab bridges, 60% of which are built before 1976. It is necessary to reassess the shear capacity of these bridges, as the traffic loads and volumes have increased significantly and the shear provisions have become more conservative. Therefore, the Dutch Ministry of Infrastructure and the Environment initiated a project to assess the shear capacity of these bridges under the live loads as prescribed by the recently implemented Eurocodes. An initial assessment indicated that 600 solid reinforced concrete slab bridges can be indicated as shear-critical. The initial assessment is based on the unity check: the ratio between the shear stress at the support due to dead load, superimposed loads and live loads as prescribed by Load Model 1 from EN 1991-2:2003 [1] and the shear capacity from NEN 6720 [2]. While no signs of distress are reported on the structures, some of the controlled cross-sections are reported to have a unity check value far above the limit value of 1 [3]. To replace all bridges in which cross-sections with a unity check value above 1 are found, would have a tremendous economic, environmental and social impact.

Improved methods to quantify the shear capacity of slab bridges are required, so that the service life of the existing structures can be prolonged. This understanding already resulted in the development of different sets of load factors for existing structures, which can be found in NEN 8700:2011 [4]. Two sets can be observed: the level "repair" and the level "replacement". For existing bridges (consequences class 3) built before 2012, the load factors at the repair level are based on a reliability index β of 3.6 [5].

1.2 Sustainable Bridges

A sustainability analysis is divided into an environmental, an economic and a social analysis [6, 7]. For buildings, rating for sustainability is common practice nowadays, but for bridges this application is rather new. When tendering a project, mostly the offers with the lowest initial costs are most successful [7]. To turn the tide, in the UK a Sustainability Index for Bridges is developed [8]. When assessing a structure for sustainability based on a life cycle analysis, five stages need to be considered: (1) the product stage, (2) the construction process, (3) the use stage, (4) the end of life stage and (5) the stage called "supplementary information beyond the building life cycle", containing benefit and loads beyond the system boundary [7]. In bridges, the operation phase plays a much less important role than in buildings, and therefore the relative importance of the construction stage and end of life stage increases. During the end of life stage, the impact is determined by the deconstruction processes, transports and finally waste processing for reuse, recovery and recycling. When the service life of a large number structures is prolonged, as can be done in the case of the existing bridges in the Netherlands when an improved method to assess the shear capacity is available, the end of life stage can be delayed. It must be pointed out that, when assessing a bridge for sustainability, different bridge and construction process components need to be taken into account: the foundation and abutments, superstructure, piles, bearings and expansions joints and temporary installations. For the existing slab bridges in the Netherlands, only the data of the superstructure is available and therefore this paper will only discuss aspects related to the superstructure.

1.3 Applying experimental results

The improvements in the shear assessment method that can lead to prolonging the service life of the structure, are applicable to concentrated loads on slabs. It is shown [9] that these loads contribute for 30% to 60% to the overall shear stress at the support. Taking into account transverse load redistribution therefore can have a significant influence on the resulting shear rating of a structure. To apply the results from experiments for a code extension formula, the safety philosophy at the considered code should be followed. If the extension is based on experimental results, the variability of the material and the ratio of the experimental results to the predictions can be described by a distribution function. To fulfill a safety requirement, expressed as a required reliability index, Monte Carlo simulations can be used. Following this strategy, an extension of the shear provisions from the Eurocode is developed.

2. BACKGROUND TO SHEAR

2.1 Transverse Load Redistribution

The shear capacity of slab bridges is calculated as the shear capacity of a beam with a large width. However, for slab bridges under concentrated loads transverse load redistribution is of particular interest. Transverse load redistribution can be taken into account by using an enhancement factor for slabs under concentrated loads in shear, and by defining the effective width in shear at the support. Theoretically, the effective slab width is determined in such a way that the shear force due to the total shear stress over the support width equals the shear force due to the maximum shear stress over the effective width. For design purposes a method of horizontal load spreading is chosen, resulting in the effective width b_{eff} at the support. The method of horizontal load spreading depends on local practice. In Dutch practice horizontal load spreading is assumed under a 45° angle from the center of the load towards the support (Fig. 1a), in French practice [10] under 45° from the far corners of the loading plate (Fig. 1b).

To study the existing data from experiments, a database of 206 experiments on wide beams and slabs [11] is compiled. The 45 experiments of slabs under concentrated loads with $b < b_{eff2}$ (a requirement to guarantee that transverse load redistribution can be enabled) with $a_v/d_l < 2.5$ (a_v is the face-to-face distance between the load and the support and d_l the effective depth to the longitudinal reinforcement) demonstrate a higher shear capacity for slabs under concentrated loads close to the support than beams [12]. However, as the majority of these experiments result from small specimens (d_l < 15cm) and from different series of experiments, there is a need for a comprehensive series of experiments.

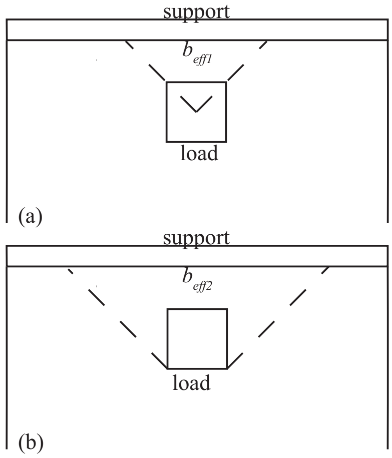

Fig.1 Effective width (a) assuming 45º horizontal load spreading from the center of the load: b_{eff1}; (b) assuming 45º horizontal load spreading from the far corners of the load: b_{eff2}; top view of slab

2.2 Limit State Function

The shear capacity of an element without shear reinforcement is determined by EN 1992-1-1:2005 [13] as:

$$V_{Rd,c} = \left[C_{Rd,c} k \left(100 \rho_l f_{ck}\right)^{1/3} + k_1 \sigma_{cp} \right] b_w d_l \geq (v_{min} + k_1 \sigma_{cp}) b_w d_l \quad (1)$$

$$k = 1 + \sqrt{\frac{200}{d_l}} \leq 2.0 \quad (2)$$

where,

k : the size effect factor
ρ_l : the longitudinal reinforcement ratio
f_{ck} : the characteristic cylinder compressive strength of concrete (in MPa)
k_1 : 0.15
σ_{cp} : the axial stress, positive in compression
b_w : the web width, to be replaced by the effective width for slabs
d_l : the effective depth to the longitudinal reinforcement.

According to the Eurocode procedures, the values of $C_{Rd,c}$ and v_{min} may be chosen by the countries involved. The default values are $C_{Rd,c} = 0.18/\gamma_c$ with $\gamma_c = 1.5$ in general and v_{min} (with f_{ck} in MPa):

$$v_{min} = 0.035 k^{3/2} f_{ck}^{1/2} \quad (3)$$

This semi-empirical expression for the shear capacity $V_{Rd,c}$ is based on a statistical analysis [14]. Therefore, an extension of the shear formula for the case of a slab under a concentrated load close to the support should be based upon a similar statistical analysis, and satisfy the same requirements with regard to the failure probability. For this purpose, the ratio between the experimental and predicted values based upon the beam shear formula of EN 1992-1-1:2005 is treated as a random variable in a statistical analysis. In a reliability analysis the variability of the loads and elements of the resistance are studied. However, when analyzing experimental results to extend a code formula, a different approach is required. To compare the experimental results to the design shear capacity from EN 1992-1-1:2005 [13], the approach used to determine the factor for the bending moment resistance of steel beams [16, 17] is used as an inspiration. The ratio between the experimental results and the predicted capacities based on the formula from EN 1992-1-1:2005 [13], Eq. 1 is treated as a random variable. The background and complete description of this procedure can be found in the full report [15]. The limit state function is used to define the failure criterion. Failure occurs when the limit state function $g < 0$, or when the load exceeds the resistance. For the case of comparing experimental results to a design method, the probability is expressed as seeking the chance that the experimental resistance is smaller than the design shear resistance:

$$P_f = P\left\{R < R_d\right\} \tag{4}$$

where,

P_f : the probability of failure
R : the experimental shear resistance
R_d : the design shear resistance.

For the comparison with the test data, the following expressions are used for R and R_d :

$$R_d = C_{Rd,c} k \left(100 \rho_{ly} f_{ck}\right)^{1/3} b_w d_l \tag{5}$$

$$R = \frac{\text{Test}}{\text{Prediction}} \times C_{Rd,c,test} k \left(100 \rho_{ly} f_{cmean}\right)^{1/3} b_w d_l \tag{6}$$

In which,

$C_{Rd,c}$: 0,12
$\frac{\text{Test}}{\text{Prediction}}$: the ratio between the experimental results and the predicted shear capacities according to EN 1992-1-1:2005 [13]
$C_{Rd,c,test}$: 0,15 for the comparison with mean values of test data [14]
f_{cmean} : the mean cylinder concrete compressive strength.

The governing criterion for the limit state is thus:

$$\left[\frac{\text{Test}}{\text{Prediction}} C_{Rd,c,test} \left(f_{cmean}\right)^{1/3} - \frac{C_{Rd,c}}{\gamma_c} \left(f_{ck}\right)^{1/3}\right] < 0 \tag{7}$$

3. EXPERIMENTAL RESULTS

3.1 Description of Experiments

Experimental research on a half-scale model of a continuous solid slab bridge was carried out at Delft University of Technology [18]. Slabs of 5m × 2.5m × 0.3m and slab strips of 5m × 0.3m with a variable width were tested. A top view of the setup with a slab of 2.5m wide on line supports is given in Fig. 2. The displacement-controlled concentrated load was placed in the middle of the width and near the edge, and close to the simple (sup 1 in Fig. 2) or continuous support (sup 2 in Fig. 2) at a variable distance to the support. In a second series of experiments, a force-controlled line load was placed at 1.2m from the support to test the specimens under a combination of loads. A line support or support consisting of 3 or 7 elastomeric bearings was used. A complete description of the experiments and measurements is given in the full test report [19, 20]. To study the influence of the major shear-defining parameters, 18 slabs were tested under a concentrated load close to the support, 8 slabs under a combination of loads and 12 slab strips under a concentrated load.

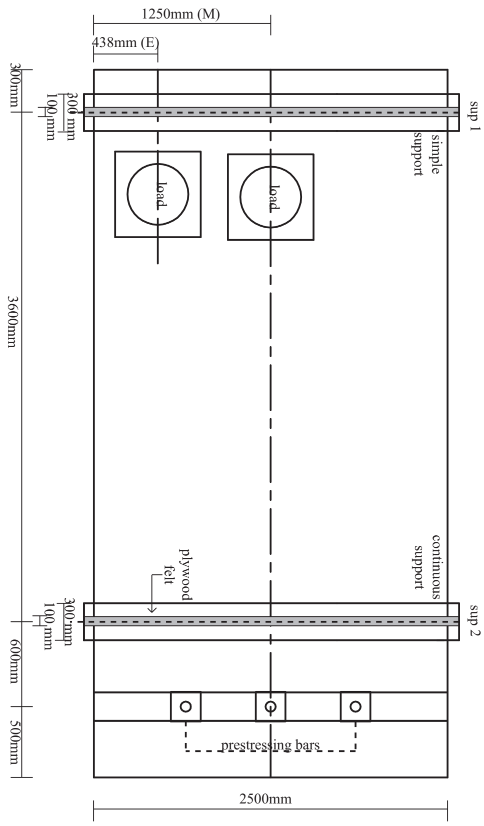

Fig.2 Top view of test setup for slabs under a concentrated load supported by a line support.

3.2 Assumed Distribution

In Eq. (7), the following random variables can be distinguished:

- Test/Prediction
- f_{cmean}
- f_{ck}

The distributions of these random variables are based on the experimental results and on the guidelines of the JCSS Probabilistic Model Code [21].

The variability of the concrete compressive strength of the mixture used for the slabs is determined from experimental results. In the laboratory, cube compressive tests are carried out at 28 days for C28/35 and C55/65. According to the JCSS Probabilistic Model Code [21], the cube concrete compressive strength follows a lognormal distribution. Thus, the experimental results of the cubes at 28 days are analyzed to determine the input values λ and ε for a lognormal distribution [22]. The histogram of the cube compressive strength for C28/35 at 28 days is shown in Fig. 3. To find the distribution of the characteristic concrete compressive strength, it is assumed that $f_{ck} = f_{c,mean} - 8$MPa as used in EN 1992-1-1:2005 [13].

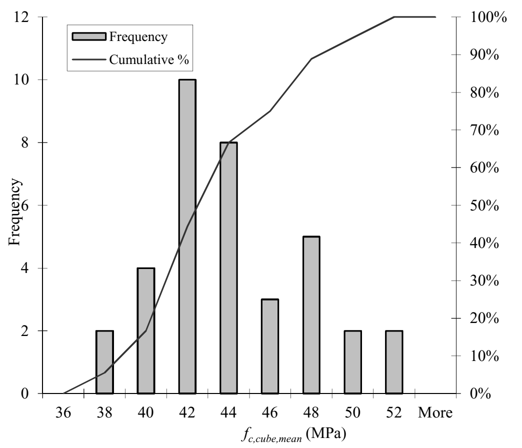

Fig. 3 Distribution of cube compressive strength for C28/35: histogram resulting from 36 cubes tested at 28 days.

The results of the ratio between the experimental results and the predictions according to EN 1992-1-1:2005 [13] are used to determine the appropriate distribution function for specimens S1 to S6 that form the reference subset of slabs with normal strength concrete, deformed reinforcement bars and supported by line supports. The French load spreading method with a minimum effective width of $4d_l$ is used to find the predicted capacity [23]. To find distributions other than the normal distribution, the following input parameters are used:

μ = 2.023: the mean value of the experimental shear forces to the predicted shear forces
σ = 0.259 : the associated standard deviation
m = 2.025 : the median of the experimental over predicted values
γ_1 = 0.098 : the skewness of the distribution
γ_2 = 0.483 : the kurtosis of the distribution
λ = 0.697 : the mean value of the natural logarithm of the ratio between the experimental and predicted shear forces
ε = 0.130 : the associated standard deviation.

The following possible distributions [22] are studied:

- a lognormal distribution, with input parameters λ and ε
- a Frechet distribution, with constants u and k that can be found from the given mean μ and standard deviation σ
- a generalized extreme value distribution, with 3 constants (a scale factor, shape factor and location constant) that can be found from the mean μ, variance σ^2 and the median m
- a Gumbel distribution, with the mode of the distribution u and the measure of the dispersion α that can be found from the mean μ and the variance σ^2
- a Beta distribution, with 4 constants (a and b limiting the interval on which the general beta distribution is defined; and q and r defining the shape of the distribution) that can be found with the mean μ, the standard deviation σ, the skewness γ_1 and the kurtosis γ_2
- a normal distribution, defined by the mean μ and the standard deviation σ.

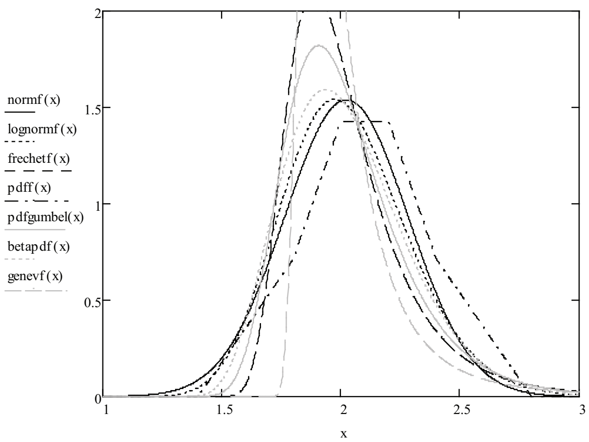

Fig. 4 Probability density function of the experimental over the calculated value (Test/Predicted), for a normal distribution, lognormal distribution, Frechet distribution, the measured distribution, the generalized extreme value distribution, the Gumbel distribution and the Beta distribution.

These distributions are then compared to the probability density function resulting from the histogram of the ratio between the test and predicted results. The results of the comparison for the probability density function are given in Fig. 3 and for the cumulative distribution function in Fig. 4. As the failure probabilities are governed by the results in the left tail, the 5% lower bounds of the ratio between the experimental results and the predictions are more closely studied to determine the most suitable distribution. The results in Fig. 3 and Fig. 4 show that the left tail is best described by a lognormal distribution or a beta distribution. The lognormal distribution is more conservative than the data from the real distribution, while the beta distribution slightly overestimates the capacity of the 5% lower bound. Therefore, the lognormal distribution will be used for the ratio between the experimental and the predicted value according to EN 1992-1-1:2005 [13] in the Monte Carlo simulations.

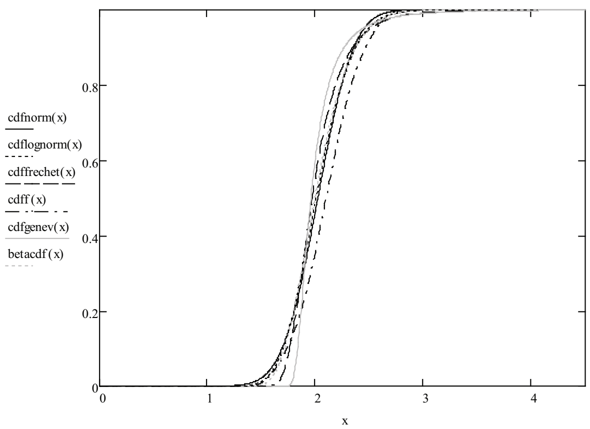

Fig. 5 Cumulative distribution function of the experimental over the calculated value (Test/Predicted), for a normal distribution, lognormal distribution, Frechet distribution, the measured distribution, the generalized extreme value distribution and Beta distribution.

4. CODE EXTENSION PROPOSAL

4.1 Results from Simulations

The required reliability level is expressed as $\alpha\beta$ = 2.88, with α = 0.8 for deterministic loading and β = 3.6. As the amount of random variables studied in the simulations is limited, the application of importance sampling techniques is not necessary. The number of samples (N) required for a Monte Carlo simulation equals [24]:

$$N > \frac{3}{P_f} \tag{8}$$

For the case where β = 3.6, the number of samples therefore is N > 18856. Therefore, the goal of the simulations is to run 10^5 trials.

To quantify the enhancement due to transverse load redistribution on the safety of slabs as compared to beams, an enhancement factor ξ on the design shear capacity is sought. The different values that are studied for the enhancement factor ξ with their resulting reliability index $\alpha\beta$, probability of failure P_f and number of trials are given in Table 1. The resulting required enhancement factor is determined also for the assumption that the ratio between the experimental results and the predictions based on EN 1992-1-1:2005 [13] follows a normal distribution. Comparing the resulting enhancement factor assuming a lognormal distribution for this ratio (1.76) to the resulting enhancement factor assuming a normal distribution (1.71) shows the importance of choosing a distribution that more closely describes the left tail of the distribution. The results lead to the conclusion that for basic cases of slabs under a concentrated load near to the support from the reference subset (normal strength concrete, deformed reinforcement bars, line supports) the design shear capacity can be increased with a factor ξ = 1.76.

Table 1 Overview of results from simulations with different enhancement factors and different distributions

Distribution	ξ	P_f	β	n
Lognormal	1	0	∞	10^5
	1.25	0	∞	10^5
	1.5	6×10^{-5}	3.8461	2×10^5
	1.6	1.98×10^{-4}	3.55	5×10^5
	1.75	0.0013	3.0115	10^4
	1.78	0.0022	2.8096	10^5
	1.77	0.0023	2.8283	10^5
	1.76	0.0020	2.8829	10^5
Normal	1.75	0.0024	2.8202	10^4
	1.73	0.0025	2.8057	10^5
	1.71	0.0019	2.9027	10^5

4.2 Code Extension Proposal

To apply the proposed enhancement factor ξ = 1.76 outside of the scope of the first subset of results (normal strength concrete, deformed reinforcement bars, line supports), the experimental results of experiments from a different subset are used to evaluate and, if necessary, alter the enhancement factor ξ. The experiments show that the influence of the concrete compressive strength [25] was not measured to influence the shear capacity as reflected by EN 1992-1-1:2005 [13], which considers the shear capacity to be proportional to the cube root of the characteristic cylinder compressive strength, Eq. (1). The enhancement factor needs to be reduced for higher strength concrete. For this purpose, the subset with results from slabs with concrete C55/65 is used. Again, a lognormal distribution is assumed for the ratio of the experimental result to the predicted value from EN 1992-1-1:2005 [13]. After several iterations [15], the enhancement factor is found to be 1.64 for concrete class C55/65. To expand the application of the enhancement factor to other concrete classes, the enhancement factor is written as a function of the characteristic concrete compressive strength f_{ck}, as used for design purposes. As only two concrete classes are tested in the experiments, a linear dependence of the enhancement factor on f_{ck} is assumed. The resulting relation based on linear interpolation is (f_{ck} in MPa):

$$\xi(f_{ck}) = 1{,}884 - \frac{f_{ck}}{225} \tag{9}$$

Next, the influence of plain bars on the enhancement factor is studied. For this purpose, a subset with the results from slabs reinforced with plain bars is compared to the results from the reference subset (deformed bars). Again, Monte Carlo simulations are used to determine the enhancement factor on the design shear capacity assuming a lognormal distribution for the results of the ratio between the experimental results and the predicted values based on EN 1992-1-1:2005 [13]. Following the same requirements for the reliability index, the enhancement factor is found to be 1.82. For the code extension proposal, it is suggested not to distinguish between slabs with plain bars and deformed bars. While the code extension proposal is aimed at existing structures, some of which are reinforced with plain bars, it should be applicable to a wide range of cases of existing structures. Therefore, it suffices to note that the proposed enhancement factor leads to safe results for slabs with plain bars. It is interesting to point out that an improved shear capacity has been observed in beams reinforced with plain bars [26] as well.

Finally, the results of slabs supported by elastomeric bearings are studied in a subset. The support length of these slabs is 1.05m per side, only 42% of the full width of 2.5m. It is observed that the reduction in the shear capacity is less pronounced than the reduction in the support length, yet significant enough to be studied in further detail through a probabilistic analysis. Assuming that only the supported length contributes to the effective width at the support is thus too conservative, and a linear expression to reduce the effective width for a reduction in the support length needs to be determined. Again, a lognormal distribution is used for the ratio between the experimental results and the predicted value based on EN 1992-1-1:2005 [13]. Through Monte Carlo simulations, it is found that the enhancement factor for this subset is ξ = 1.23, indicating an important reduction of the shear capacity as the supported length is decreased. The linear expression to reduce the effective width for a reduction in the support length is expressed in terms of the reduction factor λ on the effective width b_{eff2} resulting from the French load spreading method (Fig. 1b). The expression for λ, based on linear interpolation, is:

$$\lambda\left(\frac{l_{sup}}{b}\right) = 0.52\frac{l_{sup}}{b} + 0.48 \tag{10}$$

where

l_{sup} : the support length
b : the full width of the slab.

The proposed formula for the shear capacity of slabs under a concentrated load close to the support can be summarized as:

$$V_{Rd,c,prop} = C_{Rd,c}k\left(100\rho_l f_{ck}\right)^{1/3} b_{eff,red} d_l \left(1.9 - \frac{f_{ck}}{225}\right) \tag{11}$$

$$b_{eff,red} = \left(0.52\frac{l_{sup}}{b} + 0.48\right) b_{eff} \tag{12}$$

The results from the second series of experiments on slabs under a combination of loads [20] are used to verify the proposed formula in Eqs. (11) and (12). The enhancement factor is only to be applied to the concentrated loads on slabs, and therefore the inverse of the enhancement factor is applied as a reduction factor on the contribution of the concentrated loads to the shear stress at the support. For these experiments, concrete class C28/35 is used and the support length is 2.45m (7 bearings of 280mm × 350mm per side). Thus, the value of ξ equals 1.74. A set of 5×10^5 simulations with the assumptions of the described approach results in a probability of failure P_f = 2.08 and thus a reliability index β = 2.87, virtually identical to the required $\alpha\beta$ = 2.88. This result shows that the approach can be deemed satisfactory and the method can be used.

Finally, an extension of the formula from Eqs. (11) and (12) for concentrated loads farther away from the support ($a_v > 2.5d_l$) is sought based on the experimental results from the slab shear database [11]. To gather relevant data for a subset from the slab shear database, the following filters are applied onto the database:

- results for $a_v/d_l > 2.5$
- results for C20/25: the bounds for $f_{c,mean}$ are determined to be 20MPa – 36MPa
- $b_{eff2} < b$ to ensure that transverse load redistribution can be activated.

In total only 13 experiments satisfy the filter criteria. As can be seen in Fig. 6, these results do not appear to form a suitable subset with a distribution of the ratio between the experimental results and the predictions based on EN 1992-1-1:2005 [13]. There is a lack of experimental data to extend the formula to concentrated loads with $a_v > 2.5d_l$. Therefore, the enhancement factor cannot be used for slabs under concentrated loads with $a_v > 2.5d_l$.

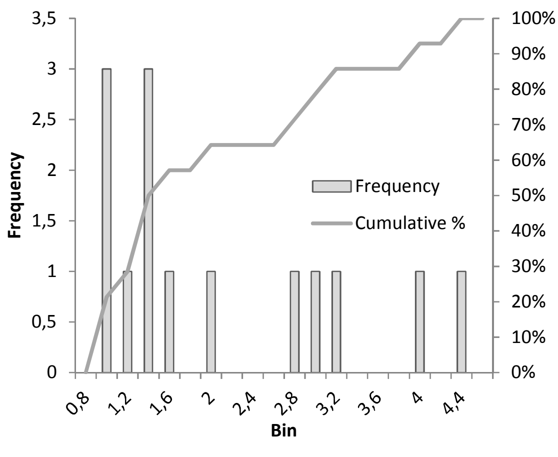

Fig. 6 Histogram of subset of data from slab shear database to verify if the code extension proposal can be used for $a_v > 2.5d_l$

While the procedure for the code extension proposal is illustrated based on the expression for the shear capacity of EN 1992-1-1:2005 [13], a similar approach can as well be followed based on other codes and design methods.

5. SUSTAINABILITY IMPACT

To assess the impact of prolonging the service life of slab bridges in the Netherlands on sustainability, the economic impact, environmental impact and impact on society should be addressed. Additional attention to the influence on climate change and use of resources are considered in the Sustainability Index for Bridges [8]. As the current study comprises a group of about 600 structures, the assessment of the impact on sustainability can only be discussed in qualitative terms. More detailed calculations of the individual structures can then determine which structures can remain in service, and lead to a quantification of the impact of prolonging the service life of this specific structure. Again, it should be noted that not all attributes can be described by a quantitative system [8]. The challenge for bridge owners and designers therefore lies in combining methodologies from different disciplines and weighing their importance before opting for a certain repair or replacement scheme.

The economic impact of prolonging the service life of a slab bridge depends on the repair costs, or alternatively the end-of-life costs combined with the cost of replacing the respective bridge. For an idea of the impact of replacing one slab superstructure, an existing slab bridge from the Netherlands [9] is considered. The superstructure is a three-span continuous slab with an end span of 10m and a mid-span of 13m. The cross-sectional depth is 0.5m and the width is 19.2m. Assuming a cost of 800 – 1000 €/m^2 leads to an initial cost of 500000 - 640000€ for the concrete deck only. The environmental impact is related to the avoidance of the environmental impact associated with repairing the existing structure or building a new structure. Revisiting the three-span solid slab bridge, and studying the impact of the 320m^3 reinforced concrete deck from the superstructure requires assessing the carbon footprint of 91 tons of concrete and 30 tons of steel. The calculated associated fossil CO_2 emission equals 136 tons based on the Carbon Calculator for Construction Activities [27]. The social dimension comprises a large number of aspects such as visual impact, time delays, job opportunities and more [28]. The impact is mostly quantified based on the driver delay costs that are associated with the refurbishment of the existing structure or with the demolition of the existing structure and placement of the new structure. These costs depend on the location of the structure and should be studied case per case. It is however very important to study these effects, as it is shown [28] that the external costs can exceed the direct costs by far. For the case studied described in [28], the social impact due to delay costs was about 9 times higher than the direct costs in the construction phase.

6. CONCLUSIONS

(1) Prolonging the service life of existing bridges requires suitable methods for assessment, such as defining different safety levels, eg. “repair” and “replacement”.

(2) To take sustainability aspects for bridges into account, the most important phases in the life cycle of the structure are the construction process and the end of life stage. Savings on the end of life stage can be realized by prolonging the service life of the structures.

(3) Transverse load redistribution leads to increased shear capacities for slabs under concentrated loads that should be accounted for when assessing a slab bridge for shear.

(4) The results of a series of experiments from Delft University of Technology are statistically analyzed and used to propose a code extension for slabs under a concentrated load close to the support benefiting from transverse load redistribution:

$$V_{Rd,c,prop} = C_{Rd,c} k \left(100 \rho_l f_{ck}\right)^{1/3} b_{eff,red} d_l \left(1.9 - \frac{f_{ck}}{225}\right)$$

$$b_{eff,red} = \left(0.52 \frac{l_{sup}}{b} + 0.48\right) b_{eff}$$

(5) Prolonging the service life of an existing structure has a direct economic impact by saving costs on repair or replacement. The ecological impact is associated with the fossil CO_2 emission of the materials required for repair or replacement. The social impact of repair or replacement is vast and can result in a multiple of the direct economic costs.

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