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Case Study on Aggregate Interlock Capacity for the Shear Assessment of Cracked Reinforced Concrete Bridge Cross-sections

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Abstract

A 55-year-old bridge showed large cracking in the approach bridge due to restraint of deformation and support settlement. After repair, it was uncertain at which crack width the traffic loads on the bridge should be further restricted. The shear capacity was calculated by counting on the aggregate interlock capacity of a supposedly fully cracked cross-section. An aggregate interlock relation between shear capacity and crack width based on an unreinforced section was used to find the maximum allowable crack width. Limits for crack widths at which load restrictions should be imposed were found. The large structural capacity of the cracked concrete section shows that the residual bearing resistance based on the aggregate interlock capacity of reinforced concrete slab bridges with existing cracks is higher than expected. This expected capacity could be calculated with the inclined cracking load from the code provisions. The procedure outlined in this paper can thus be used for the shear assessment of fully cracked cross-sections of reinforced concrete bridges.

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Introduction

As a result of increased traffic loads and volumes, the demands on reinforced concrete slab bridges that were built several decades ago, are higher than calculated using the governing codes at the time of design. At the same time, the shear capacities prescribed by the currently governing NEN-EN 1992-1-1:2005 (CEN 2005) and the Dutch National Annex NEN-EN 1992-1-1+C2:2011/NB:2011 (Code Committee 351 001 2011) are smaller than allowed in the previously used national code. This combination of smaller prescribed shear capacities and larger live loads led to a situation in which the shear capacity of 600 of the existing solid slab bridges in the Netherlands is under discussion.

For assessment of slab bridges, an approach based on Levels of Approximation was developed. Levels of Approximation are part of the solution method that is used in the *fib* Model Code (fib 2012). A Level of Approximation I provides a fast but conservative solution. As the Level of Approximation is increased, the computed result more rigorously estimates the capacity of the structural element, but the elapsed time and effort increase, see Figure 1. In the *fib* Model Code, the shear and punching capacity is calculated based on different Levels of Approximation. A similar approach is followed as well in the Netherlands for the shear assessment of existing bridges. If a lower Level of Approximation shows that the considered bridge cross-section has sufficient capacity, then no further calculations need to be done. If an insufficient capacity is found, the analysis is repeated with a higher Level of Approximation, to have a more precise estimate of the capacity. For shear assessment of slab bridges, three Levels of Approximation

can be distinguished. Level of Approximation I is the so-called "Quick Scan Method" for shear (Lantsoght et al. 2013b; Lantsoght et al. (in press, b)). This method is programmed in a spreadsheet, in which the entire database of cross-sections can be evaluated at once. The result of the Quick Scan method is the Unity Check of the considered cross-sections. The Unity Check is the ratio of the shear stress at the support due to dead load, superimposed load and live loads over the shear capacity. If the Unity Check value of one of the considered cross-sections is larger than 1, a further analysis of the structure is required. Level of Approximation II means using a linear finite element program, in which the peak shear stress at the support, distributed over a distance $4d_l$ (Lantsoght et al. 2013a), is compared to the prescribed shear capacity. If the cross-section still proves to be insufficient, Level of Approximation III can be used for further, typically probabilistic analysis. Then, in Level IV either a non-linear finite element analysis can be used, or the shear capacity can be estimated based on one of the shear-carrying mechanisms, such as aggregate interlock. Determining the shear capacity of a cracked cross-section of a solid slab bridge is the topic of this paper.

Although typically slab bridges are calculated as beams with a large width, research has been done to investigate the behavior of this bridge type, indicating sources of residual capacity in reinforced concrete solid slab bridges (Aktan et al. 1992; Azizinamini et al. 1994a; Azizinamini et al. 1994b). In slab bridges, the main source of residual capacity is the slab's transverse load redistribution capacity (Lantsoght et al. (in press, a)). Taking sources of additional capacity into account leads to a better estimate of the bearing capacity, even to such an extent that retrofitting might become unnecessary. Walraven (Walraven 2010) demonstrated that determining the shear bearing capacity should not be done with design equations derived for new structures from building codes. As such, using a shear-carrying mechanism such as aggregate

interlock that is well-understood and that has been quantified through models and experiments, is a valid option for carrying out the assessment of structures that need further analysis (Level of Approximation IV).

This paper deals with the shear assessment of a 55-year-old reinforced concrete slab bridge in The Netherlands with extensive cracking in the southern concrete approach bridge. The aggregate interlock capacity of the cracked section was used to estimate the residual shear capacity.

Description of bridge

Geometry and Support Conditions

The bridge under study consists of two concrete approach bridges and a moveable steel bridge crossing a canal. Major cracking was observed in the southern concrete approach bridge, which is further studied in this paper. The southern approach bridge consists of a three-span continuous bridge LMNO and a four-span continuous bridge OPQRS. The fixed support lines are at beams N and Q and the joints are at beams O, L and S (Fig. 2a).

Material Properties

Based on core samples (locations as shown in Figure 2a) the concrete strength was determined as a Dutch class B45 (equal to C35/45 from NEN-EN 1992-1-1:2005 (CEN 2005), with a characteristic cylinder compressive strength f_{ck} = 35MPa). Plain reinforcement bars of type QR24 were used. According to the guidelines for existing bridges of the Dutch Ministry of Infrastructure RBK (Rijkswaterstaat 2013) the design value of the tensile strength of this type of steel is f_{yd} = 209 MPa and the strain at failure is ε_{su} = 5%. According to the Dutch Reinforced

Concrete Recommendations from 1950, "GBV 1950" (Royal Institute of Engineers 1950) (as used to design the considered bridge) the strain at failure is at least ε_{su} = 27% and research (Den Uijl 2004) has shown that the ultimate strain of reinforcing bars taken from existing bridges from the 1960s is between 19% and 38%. An ultimate strain of 5% is therefore a conservative value.

Cracking in span NM

The height of the cross-section in the slab LMNO varies from 500 mm (side) to 580 mm (middle of the slab). The reinforcement drawings show that the bottom longitudinal reinforcement consists of 14 mm bars with a center-to-center spacing of 200 mm ($\rho_{l,bot}$ = 0.154%) and that the top reinforcement consists of 25 mm bars with a spacing of 100 mm ($\rho_{l,top}$ = 0.993%). The transverse flexural reinforcement consists of 14 mm bars with a spacing of 200 mm for the top ($\rho_{t,top}$ = 0.163%) and of 150 mm for the bottom ($\rho_{t,bot}$ = 0.212%). The transverse flexural reinforcement is only 16% of the longitudinal reinforcement for the top bars, which is less than the recommended value of minimum 20%.

An overview of the damage observed in the southern approach bridge is given in Table 1. The cross-sections near a given support of a given span are analyzed one by one. The column "Crack?" indicates whether or not structurally significant cracks are observed at that location. The column "Type" then identifies the type of crack(s), and the column "Width" gives the maximum measured crack width in mm at that location.

The largest observed crack (Table 1) was a flexural crack in span NM (Fig. 2a), most likely caused by a support settlement. The effect of the settlement was taken away by jacking up the support to its original position. It was measured on site that beam N was jacked up 21 mm on the west side (Fig. 3). This height was linearly reduced to 0 mm on the east side.

Due to the large settlement, the flexural reinforcement reached its yield stress at the location of the crack. However, the safety of the bridge was not compromised thanks to the large ductility and high failure strains of the plain reinforcement that was used in the bridge.

In the vicinity of the supports, the amount of flexural reinforcement at the bottom of the section does not satisfy the requirements (minimum 50% of the bottom reinforcement used at midspan) of both the recent Dutch code provisions NEN 6720:1995 (NEN Committee 351001 1995) and the 1950s provisions GBV 1950 (Royal Institute of Engineers 1950). As a result, the construction is vulnerable to the deformation due to the restraints that can occur as a result of support settlements and stresses induced from changes in temperature, as the rusted steel bearings do not allow for movement. The amount of longitudinal bottom reinforcement is also smaller than the required amount of 0.21% from NEN 6720:1995 (NEN Committee 351001 1995) for concrete class B45. Failure of the cross-section will result from breaking of the steel reinforcement before crushing of the concrete.

Cracking in spans PQ and RQ

The height of the cross-section in OPQRS varies from 450 mm at the side (with an effective depth to the longitudinal reinforcement d_l = 413 mm) to 530 mm in the middle (with d_l = 493 mm). The reinforcement drawings show that only 14 mm bars with a center-to-center distance of 200 mm are present and continue 1.25 m past the support ($\rho_{l,bot}$ = 0.156%) and the top reinforcement consists of 25 mm bars at 100 mm center-to-center ($\rho_{l,top}$ = 1.007%). The transverse flexural reinforcement consists of 14 mm bars with a spacing of 200 mm for the top ($\rho_{t,top}$ = 0.181%) and of 150 mm for the bottom ($\rho_{t,bot}$ = 0.236%). The transverse flexural

reinforcement is only 18% of the longitudinal reinforcement for the top bars, which is less than the recommended value of minimum 20%.

In the bridge part OPQRS flexural cracks as well as through cracks over the entire depth are observed (Fig. 4). A typical flexural crack as occurs in a reinforced concrete structure is shown in Figure 4a. The type of crack which runs through the complete cross-section, caused by axial tension, is shown in Figure 4b and c. For identical top and bottom reinforcement and for uniform axial tension, the crack width will be constant over the depth of the deck (Fig. 4b). The cracks through the deck observed in the bridge OPQRS are estimated to be as shown in Figure 4c because the amount of top reinforcement (support reinforcement) is larger than the amount of bottom reinforcement. The type of crack from Figure 4c can also be caused by a combination of bending moment and axial tension, where the largest crack width corresponds to the side of the cross-section with the largest tensile stresses. For the considered case, however, the effect of axial tension is considered to be dominant. Unfortunately, the crack width can only be measured at the bottom side of the deck because the wearing surface obstructs the inspection of the crack width at the top face of the cross-section. When the bottom reinforcement has yielded but the top reinforcement has not yielded yet, the measured crack width at the bottom side of the deck will be considerably larger than the crack width at the top of the deck. The latter crack width then has to be calculated. Upon yielding of the reinforcement, dowel action can also lead to a relative vertical displacement of the crack faces. In the case of significant yielding of the reinforcement, it is thus recommended to remove the asphalt layer and measure the crack width at the top of the cross-section as well.

In span PQ, a crack caused by the combination of restraint of deformation due to rusted steel bearings and the traffic load was observed at 1.3 m from girder P (Fig. 2a) and in span RQ a

similar crack was observed at 1 to 1.3 m from girder R (Fig. 2a). At the positions where cores had been drilled out of the slab, a maximum crack width of 0.7 mm was measured (Table 1). It was observed that the crack ran along the aggregates; therefore, the aggregate interlock capacity was safeguarded.

Repair actions and current situation

The cracks have been injected with epoxy and the support was jacked back to its original elevation, which resulted in a stable situation in the cracked reinforced concrete deck. The traffic over the bridge is restricted to pedestrians, bikes, cars and buses. Heavy trucks are not allowed. A monitoring program, consisting of measuring the cracks every 4 weeks and regular visual inspection of the bridge, is in place. Replacement of the bridge is scheduled for the near future.

Aggregate interlock

What is aggregate interlock?

Aggregate interlock is one of the shear resisting mechanisms of structural concrete. Because the strength of the hardened cement paste in most concretes is lower than the strength of the aggregate particles, cracks intersect the cement paste along the edges of the aggregate particles. So the aggregate particles, extending from one of the crack faces, “interlock” with the opposite face and resist shear displacements (Walraven 1980). The aggregate interlock shears depend on the surface roughness of the cracks, the aggregate type and the displacements across the cracks (Taylor 1974).

The fundamental model for aggregate interlock

Walraven developed a model for aggregate interlock (Walraven 1980; 1981a; Walraven 1981b) in which concrete is taken as a two-phase material consisting of stiff aggregate particles embedded in an ideally-plastic matrix. Measurements on beams had shown that cracks do not open to their final width and subsequently shear, but open and shear simultaneously. Therefore, both the shear stress and normal stress have to be taken into account as essential components. Assuming that the irregular faces of the crack can be deformed, both the shear stress τ and the normal stress σ are functions of the crack width w and the shear displacement Δ.

Walraven's fundamental model for aggregate interlock (Walraven 1981b) is based on a statistical analysis of the crack structure and the contact areas between the crack faces as a function of the displacements, w and Δ, and the composition of the concrete mix. Two fundamental modes of behavior characterize the aggregate interlock: sliding at the contact area between particles and matrix at opposite sides of the crack ("overriding') and irreversible deformation of the matrix by a high contact stress.

Considering concrete as a combination of a matrix and aggregate particles, and taking into account that the size of the particles is considerably greater than the crack width, the micro-roughness of the crack (aggregate particles projecting from the crack plane) is seen as dominant with respect to the macro-roughness (the overall undulations of the crack plane). The micro-roughness and the particles that protrude from a surface are shown in Fig. 5. The contact surface of the particles with particles from the other side of the crack is highlighted in grey.

Initially, the contact areas tend to slide, so that the contact area is reduced. This leads to high contact stresses, resulting in plastic deformations until in x- and y-direction equilibrium of

forces is obtained. On the contact area, the stresses are resolved into a stress normal to the contact area σ_{pu} and tangential τ_{pu} (Fig. 6).

A rigid-plastic stress-strain relation for the matrix is used, since it is expected that the plastic deformation will be significantly larger than the elastic deformation.

To find the contact areas in the *x*- and *y*-direction for a unit crack area as a function of the displacements between both crack faces, the size distribution of the aggregates is studied. The size of the aggregates determines the probability density function of the number of intersection circles with a given diameter from the protruding aggregates that intersect the studied unit crack length. Once this function is described, the intersection circles modeling the protruding aggregates from both sides of the crack surface can be studied. The contact area of the circles from both sides then defines the contact area between the crack faces.

Experimental results from push-off tests were used to determine the matrix yielding stress σ_{pu} and the friction coefficient μ. The friction coefficient was found to be μ = 0.4 (Walraven 1981b) and

$$\sigma_{pu} = 6.39\, f_c^{'0.56} \text{ (N/mm}^2\text{)} \tag{1}$$

with f_c' the cube compressive strength of the concrete. The matrix yielding strength is slightly higher than the strength of the concrete itself, because micro-cracking at the paste-aggregate interface reduces the capacity.

For concrete with gravel aggregates (maximum aggregate size 16 mm to 32 mm) and cube crushing strengths f_c' between 13 MPa and 59 MPa, simplified linear relations were developed (Walraven 1981a) (τ, σ > 0, units N, mm):

$$\tau = \frac{-f_c^{'}}{30} + \left[1.8w^{-0.8} + \left(0.234w^{-0.707} + 0.20\right) f_c^{'}\right]\Delta \tag{2}$$

$$\sigma = \frac{-f_c^{'}}{20} + \left[1.35w^{-0.63} + \left(0.191w^{0.522} - 0.15\right) f_c^{'}\right]\Delta \tag{3}$$

In all experiments, the crack opening path was influenced by the external restraint stiffness. For a larger restraint stiffness, the crack opening path becomes stiffer.

For reinforced concrete, the mechanism works in a similar way (Walraven 1981a). The restraining force is now provided by the reinforcement and depends on the bond between reinforcement and concrete and on the yield strength. It was observed experimentally that the crack opening path does not seem to be significantly influenced by the reinforcement ratio. Assuming that the relationship between the shear stress τ_u and the normal restraining stress $\rho \times f_y$ (with ρ the reinforcement ratio and f_y the yield stress) in a reinforced crack is similar to the relation between τ_u and σ in an unreinforced crack, leads to (units N, mm):

$$\tau_u = C_1\left(\rho \times f_y\right)^{C_2} \tag{4}$$

$$C_1 = \left(f_c^{'}\right)^{0.36} \tag{5}$$

$$C_2 = 0.09\left(f_c^{'}\right)^{0.46} \tag{6}$$

Eq. (4) is based on the assumption that all flexural reinforcement in a cross-section provides a clamping force on the crack. In the case of axial tension on the cross-section, the clamping action of the reinforcement will be reduced by this tension. In the case of flexure, both internal tension (reducing the clamping force) and compression (increasing the clamping force) will occur, and the effect will be smaller than when significant axial tension is present on the cross-section. Therefore, in the following analysis, only the effect of axial tension on the clamping force is considered.

Contribution of aggregate interlock to the shear capacity

An overview of the contribution of aggregate interlock to the total shear capacity at failure as reported in the literature is given in Table 2. The results of Hamadi and Regan (1980) show that the aggregate interlock capacity depends on the type of aggregates used: weaker aggregates will result in a lower relative contribution of aggregate interlock to the total shear capacity. The aggregates used in the bridge under study were gravel aggregates from rivers. The results by Fenwick and Paulay (1968), Taylor (1972) and Kani et al. (1979) are obtained from testing small, heavily reinforced concrete beams with $a/d_l > 2.5$, which might not be directly representative for slabs. The analysis by Sherwood et al. (2007) was carried out for wide beams and slabs, indicating that aggregate interlock is the main shear carrying mechanism in wide elements.

Swamy and Andriopoulos (Swamy and Andriopoulos 1973) combined the amount of forces transferred through aggregate interlock and dowel action. They measured the contribution of aggregate interlock and dowel action to vary between almost 90% for a beam with 1.97% of tension steel and shear span-to-depth ratio $a/d_l = 2$ to about 50% for a beam with 3.95% of tension steel and $a/d_l = 6$. This result indicates that for slab bridges, containing less reinforcement than a typical beam specimen from a shear test, the aggregate interlock capacity is the major shear carrying mechanism. Assessing the shear capacity based on the aggregate interlock capacity is a conservative approach, since the effect of the other mechanisms of shear transfer is neglected.

Capacity of a cracked cross-section

Shear capacity based on code methods

According to the Dutch code NEN 6720:1995 (NEN Committee 351001 1995) the design shear capacity of a regular cross-section (without a through crack), is:

$$V_{NEN\,6720} = 0.4\, f_{ctd}\, d \times b \tag{7}$$

where

f_{ctd} = the design tensile strength of the concrete = 1.65 MPa for this case;

d = the effective depth of the considered cross-section;

b = the width (unit width of 1 m).

Equation 7 results in a shear capacity $V_{NEN6720}$ in span RQ (governing case) of 273 kN/m at the side and 325 kN/m in the middle of the considered cross-section.

Shear capacity of a section with a through crack

The shear capacity of a section with a through crack is calculated based solely on its aggregate interlock capacity:

$$V_{agg} = \tau_u \times d \times b \tag{8}$$

where τ_u = the shear stress from Equation 4. Dowel action is neglected, which is a conservative assumption. The reinforcement ratio is taken as half of the provided reinforcement ratio in the cross-section to account for the lower bond capacity of plain reinforcement, as is commonly assumed in Dutch practice. To convert Equation 4 into a design value, the result is multiplied by 0.85 / 1.35. The value of 0.85 takes into account the long-term effects of the concrete behavior. The factor 1.35 transforms the equation for average values into an equation for characteristic values. All calculations are carried out with the characteristic values of the material properties.

This approach results in an aggregate interlock capacity of 1679 kN/m. The capacity of the section with a through crack V_{agg} is considerably larger than the shear capacity $V_{NEN6720}$ of the section according to NEN 6720:1995 (NEN Committee 351001 1995). This comparison shows the large shear resistance provided by aggregate interlock action.

Maximum crack width

Since the cracks in the bridge are being monitored, the next question was at which crack width measured during inspection of span RQ, the traffic loads on the bridge should be further restricted to only pedestrians and bikes. The maximum crack width at bending failure is determined from the ultimate strain in the reinforcement. The strain in the elastic range is neglected (conservative assumption). A strain at failure of 5%, the limit from the Guidelines Existing Bridges (Rijkswaterstaat 2013), over a length equal to 5 times the diameter (A5 value from Dutch certification (OVBS-Benor 2013)) results in a crack width of 3.5 mm for a bar with diameter 14 mm. Because the existing crack was injected and the support is jacked, the capacity of the bridge deck has been partially restored to its original state. However, part of the plastic deformation capacity of the yielding reinforcement has already been used. It is then conservative to limit the maximum crack width to half of the calculated value: 1.8 mm ≈ 2 mm.

Cracks over full depth

To find a relation between the crack width w and the aggregate interlock capacity, an unreinforced section was assumed (Equation 2 and 3), in which the shear force V_{u_unr} and axial force F_{ax} are determined:

$$V_{u\_unr} = \tau \times b \times h \tag{9}$$

$$F_{ax} = \sigma \times b \times h \tag{10}$$

where

τ = shear stress as given in Equation 2;

σ = normal stress as given in Equation 3;

h = height of the cross-section; and

b = 1 m.

A constant crack width over the depth as shown in Figure 3b is assumed. It is conservative to assume that the crack width measured at the bottom of the slab is the maximum crack width, since the crack width will be smaller at the top of the section (as explained earlier) and hence the average crack width in the section will be smaller. A larger crack width is conservative because less particles protruding from the crack faces will make contact, resulting in a lower aggregate interlock capacity. Moreover, the crack width on the bottom is the only crack width of the cross-section that can be measured because of the asphalt layer on the top surface.

The relation between the shear capacity V_{u_unr} and the crack width w was used to find the crack width at which the shear capacity V_{u_unr} of the section with a through crack becomes smaller than the shear capacity $V_{NEN6720}$ of the section without a through crack according to NEN 6720:1995 (NEN Committee 351001 1995). Based on the graphs that show the relation between crack width and crack slip from Walraven (Walraven 1981a), it is assumed that for normal strength concrete with a maximum aggregate size of 32 mm the following relation between the crack width w and the shear displacement Δ can be used:

$$\Delta = 1.25 \times w \tag{11}$$

For an unreinforced section, it was found that at 1.3 mm crack width (Fig. 7a) the aggregate interlock capacity V_{u_unr} is fully lost and that at a crack width of 1.2 mm (Fig. 7a) the

shear capacity V_{u_unr} (Equation 9) of the section with a through crack becomes smaller than the shear capacity $V_{NEN6720}$ (Equation 7) of the section without a through crack.

A similar approach is followed for the axial load resulting from the normal stress on the crack considered in the aggregate interlock theory. Since the steel bearings of the bridge deck are rusted, it is conservatively assumed that these cannot allow any movement. Large axial forces will result on the cross-section due to restrained deformation as a result of temperature changes. To account for this restraint of deformation, the conservative assumption is made that the entire concrete cross-section is subjected to tension. If the entire cross-section is in tension, a resulting tensile force F_{tc} (Eq. 12) can be calculated, based on the tensile strength from NEN 6720:1995 (NEN Committee 351001 1995) as given in Equation 13. This tension force needs to be balanced by the tension in the reinforcement steel, so that less tension force remains in the reinforcement to apply a clamping force on the crack. If no tension occurs on the cross-section, the force F_{steel} from Equation 14, assuming yielding of the reinforcement, acts on the crack. When part of the tension force is needed to balance the concrete tension, a lower clamping force F_{clamp} from Equation 15 remains.

$$F_{tc} = f_{ctk} d \times b \tag{12}$$

$$f_{ctk} = 0.7\left(1.05 + 0.05 f_c^{'}\right) \text{ with } f_{ctk} \text{ and } f_c^{'} \text{ in [MPa]} \tag{13}$$

$$F_{steel} = \left(A_{s,bottom} + A_{s,top}\right) f_y \tag{14}$$

$$F_{clamp} = F_{steel} - F_{tc} \tag{15}$$

with $A_{s,bottom}$ and $A_{s,top}$ the area of the bottom and the top reinforcement in the cross-section respectively.

At a crack width of 1.3 mm (Fig. 7b) the axial force due to the restraint of deformation F_{clamp} (Equation 15) becomes larger than the axial force from aggregate interlock F_{ax} (Equation

10). This maximum crack width becomes 1.1 mm at the side of the deck where the height is reduced to 450 mm.

Axial force from restraint of deformation

In a next step, the influence of the restraint of deformation on the axial tensile capacity $N_{tension}$ of the cross-section is studied. The axial tensile capacity needs to be studied along with the aggregate interlock capacity (shear capacity), because it can be seen in Equations 2 and 3 that both the shear and axial stresses occur when a crack opens and slips. The results are summarized in Table 3, in which $N_{tension}$ is the remaining axial capacity from aggregate interlock of the cracked section and F_{tc} is the axial tensile force on the cross-section from Equation 12. The procedure for finding $N_{tension}$ is now explained.

According to NEN 3865:1977 clause E-508 (NEN Committee Concrete Structures 1977), the maximum allowable crack width in [mm] for combined flexure and tension is:

$$w_{max,NEN3865} = 0.8\sigma_a \xi_2 \left(2c + \xi_3 \frac{\phi_{top}}{\rho_{top}} \right) 10^{-5} \tag{16}$$

where

c = the concrete cover;

ξ_2 = 1.25;

ξ_3 = 2.5;

ϕ_{top} = diameter of the top reinforcement;

ρ_{top} = top reinforcement ratio (in %); and

σ_a = the tensile stress in the cross-section as a result of restraint of deformation in [MPa]; with

$$\sigma_a = \frac{F_{tc}}{A_{s,top}} \leq f_{yk} \tag{17}$$

with F_{tc} from Equation 12 and $A_{s,top}$ = the area of the top reinforcement in the cross-section.

Consequently, it is assumed that only the effective tensile area in the upper part of the slab contributes to the capacity. A fictitious tension tie inside a member subjected to bending is thus studied. The effective height (of the fictitious tension tie) of the cross-section is defined as:

$$h_{eff} = 2.5\left(c + \frac{\phi_{top}}{2}\right) \tag{18}$$

The shear force from aggregate interlock V_{agg} should be at least twice the shear-flexure capacity $V_{NEN6720}$ of the section without a through crack to account for the difference between using design values for $V_{NEN6720}$ and characteristic values for V_{agg}. A safety factor of 2 is thus built into the procedure. This requirement results in an axial force from aggregate interlock N_{agg} for the calculated crack width w and shear displacement Δ on the effective area (product of the effective height h_{eff} and a unity width b):

$$V_{agg} = \tau \times h_{eff} \times b \tag{19}$$

$$N_{agg} = \sigma \times h_{eff} \times b \tag{20}$$

with

τ = the shear stress from Equation 2;

σ = the axial stress from Equation 3;

h_{eff} from Equation 18; and

b = 1 m.

The horizontal equilibrium on the crack in the zone of the fictitious tension tie encompasses the axial force N_{agg} (Eq. 20) from aggregate interlock for a given crack width $w_{max,NEN3865}$, the tension caused by the restraint of deformation, and the clamping force provided by the steel reinforcement. As a result, the remaining capacity $N_{tension}$ available to resist the

deformation results from subtracting N_{agg} from the force in the top reinforcement assuming yield F_{top}:

$$F_{top} = A_{s,top} \times f_y \tag{21}$$

$$N_{tension} = F_{top} - N_{agg} \tag{22}$$

Both the cross-section in the middle and at the side of the deck were checked. The middle section, with a deck height of 530 mm, is governing; these values are shown in Table 3. For more than 71% of restraint, the equilibrium conditions are not met, and the external tension on the cross-section will be larger than the internal resistance against tension.

The results in Table 3 show that it is important to know the amount of restraint of deformation in the cross-section in order to be able to verify if the equilibrium conditions are met. It also shows that a check of the axial forces is necessary for a shear problem when analyzing based on aggregate interlock capacity.

Overview of maximum allowable crack widths

The maximum crack width allowed was determined to be 1 mm on average over the entire width of the deck for a new through crack in span RQ. This value is determined based on the calculations for the maximum crack width (Fig. 7), which resulted in a maximum crack width of 1.1 mm. This value has been rounded off to 1 mm.

For the repaired crack in span NM an increase in crack width of 0.5 mm is allowed. This crack was repaired by injection with epoxy, so that internal compressive stresses in the cross-section develop. Because of these internal compressive stresses (compare this to the effect of prestressing a cross-section), it is not expected that live loads will cause opening of the cracks. Only other, unexpected causes, can result in an opening of these cracks. Therefore, an increase in crack width of only 0.5 mm is allowed.

If larger crack widths are observed, the traffic should be restricted to bikes and pedestrians.

Recommendations

To take away the cause of the restraint of deformations, it was advised to replace the rusted steel bearings by elastomeric bearings. This option also ensures that the bridge can be available to all traffic and that the service life can be extended.

To quantify the amount of restraint introduced onto the section, measurements of the deformation in the joints and the temperature are proposed. These data would allow a more precise estimate of the capacity of the cracked cross-section and a verification of the axial equilibrium conditions.

Summary and Conclusions

The large structural capacity of the cracked concrete section studied in this case shows that the residual capacity based on the aggregate interlock capacity of reinforced concrete slab bridges with existing cracks is estimated to be significantly higher than the inclined cracking load used by the design codes. Even for large tensile forces on the considered cross-section, the aggregate interlock capacity remains high.

The axial equilibrium has to be verified as well, which in this case was not fulfilled for all restraint levels because of the estimated tension forces on the cross-section.

Calculating the aggregate interlock capacity of a cracked section offers a practical and easy-to-implement method to determine the residual bearing capacity of existing concrete

bridges with extensive cracking. This method is thus suitable for a Level of Approximation IV approach for shear assessment.

Notation List

The following symbols are used in this paper:

a = center-to-center distance between load and support

b = width

c = concrete cover

d = effective depth of the considered cross-section

d_l = effective depth to the longitudinal reinforcement

f_c' = cube compressive strength of the concrete

f_{ck} = characteristic cylinder compressive strength

f_{ctd} = design tensile strength of the concrete

f_{ctk} = characteristic tensile strength of the concrete

f_y = yield stress

f_{yd} = design value of the tensile strength of the reinforcement steel

f_{yd} = characteristic value of the tensile strength of the reinforcement steel

h = height of the cross-section

h_{eff} = effective height of the cross-section

w = crack width

$w_{max,NEN3865}$ = maximum allowable crack width for combined flexure and tension

x = horizontal axis

y = vertical axis

$A_{s,bottom}$ = area of the bottom reinforcement in the cross-section

$A_{s,top}$ = area of the top reinforcement in the cross-section

C_1 = parameter in aggregate interlock formulas

C_2 = parameter in aggregate interlock formulas

F = force, not otherwise specified

F_{ax} = axial capacity based on maximum normal stress in aggregate interlock theory

F_{clamp} = resulting clamping force on the cross-section

F_{steel} = clamping force on the crack assuming yield of the reinforcement

F_{tc} = resulting tensile force

F_{top} = force in the top reinforcement assuming yield

N_{agg} = axial force from aggregate interlock

$N_{tension}$ = axial tensile capacity of the cross-section

V_{agg} = shear capacity of a section based on the ultimate aggregate interlock capacity

$V_{NEN6720}$ = design shear capacity of section without through crack according to NEN 6720:1995 (NEN Committee 351001 1995)

V_{u_unr} = shear capacity from aggregate interlock of an unreinforced cross-section

ε_{su} = strain at failure of the reinforcement steel

ϕ_{top} = diameter of the top reinforcement

μ = friction coefficient

ρ = reinforcement ratio

$\rho_{l,bot}$ = reinforcement ratio for the longitudinal reinforcement on the bottom of the cross-section

$\rho_{l,top}$ = reinforcement ratio for the longitudinal reinforcement on the top of the cross-section

$\rho_{t,bot}$ = reinforcement ratio for the transverse flexural reinforcement on the bottom of the cross-section

$\rho_{t,top}$ = reinforcement ratio for the transverse flexural reinforcement on the top of the cross-section

ρ_{top} = top reinforcement ratio (in %)

σ = normal stress

σ_a = tensile stress in the cross-section as a result of restraint of deformation

σ_{pu} = stress normal to the contact area

τ = shear stress

τ_u = ultimate shear stress

τ_{pu} = stress tangential to the contact area

ξ_2 = parameter in expression for allowable crack width

ξ_3 = parameter in expression for allowable crack width

Δ = shear displacement

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List of Figures

List of Tables

Table 1. Overview of damage to southern approach bridge.

Support	Span	Crack?	Type	Width (mm)
M	MN	x	flexural	0.1 - 0.25
N	NM	x	flexural (injected)	0.6 - 0.8
N	NO	x	flexural	-
O	ON	-	-	-
O	OP	x	flexural, span-direction	-
P	PO	x	flexural, span-direction	-
P	PQ	x	flexural/through (injected)	0.7
Q	QP	-	-	-
Q	QR	-	-	-
R	RQ	x	through (injected)	0.4 - 0.7

Table 2. Contribution of aggregate interlock as percentage of total shear carrying capacity at failure.

Author(s)	Year	%	Comments
(Fenwick and Paulay 1968)	1968	60	measured
(Taylor 1972)	1972	33- 50%	measured
(Sherwood et al. 2007)	2007	< 70%	
(Kani et al. 1979)	1979	50 - 60%	
(Hamadi and Regan 1980)	1980	44%	natural gravel aggregates
		26%	expanded clay aggregates
(Swamy and Andriopoulos 1973)	1973	50 – 90%	

Table 3. Sensitivity to axial force based on percentage of restrained deformation

Restraint %	$N_{tension}$ kN/m	F_{tc} kN/m
100	793	1139
75	810	854
71	814	809
50	841	570
25	888	285