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

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4
5
6 **Abstract**

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

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1 **CE database subject headings**

2 aggregates; assessment; bridge maintenance; bridges; concrete slabs; cracking; shear
3 resistance

4
5 **Introduction**

6 As a result of increased traffic loads and volumes, the demands on reinforced concrete
7 slab bridges that were built several decades ago, are higher than calculated using the governing
8 codes at the time of design. At the same time, the shear capacities prescribed by the currently
9 governing NEN-EN 1992-1-1:2005 (CEN 2005) and the Dutch National Annex NEN-EN 1992-
10 1-1+C2:2011/NB:2011 (Code Committee 351 001 2011) are smaller than allowed in the
11 previously used national code. This combination of smaller prescribed shear capacities and larger
12 live loads led to a situation in which the shear capacity of 600 of the existing solid slab bridges in
13 the Netherlands is under discussion.

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

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

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

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

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

8

9 **Description of bridge**

10 *Geometry and Support Conditions*

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

16

17 *Material Properties*

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

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

5 6 ***Cracking in span NM***

7 The height of the cross-section in the slab LMNO varies from 500 mm (side) to 580 mm
8 (middle of the slab). The reinforcement drawings show that the bottom longitudinal
9 reinforcement consists of 14 mm bars with a center-to-center spacing of 200 mm ($\rho_{l,bot} =$
10 0.154%) and that the top reinforcement consists of 25 mm bars with a spacing of 100 mm ($\rho_{l,top}$
11 = 0.993%). The transverse flexural reinforcement consists of 14 mm bars with a spacing of 200
12 mm for the top ($\rho_{t,top} = 0.163\%$) and of 150 mm for the bottom ($\rho_{t,bot} = 0.212\%$). The transverse
13 flexural reinforcement is only 16% of the longitudinal reinforcement for the top bars, which is
14 less than the recommended value of minimum 20%.

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

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

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

4 In the vicinity of the supports, the amount of flexural reinforcement at the bottom of the
5 section does not satisfy the requirements (minimum 50% of the bottom reinforcement used at
6 midspan) of both the recent Dutch code provisions NEN 6720:1995 (NEN Committee 351001
7 1995) and the 1950s provisions GBV 1950 (Royal Institute of Engineers 1950). As a result, the
8 construction is vulnerable to the deformation due to the restraints that can occur as a result of
9 support settlements and stresses induced from changes in temperature, as the rusted steel
10 bearings do not allow for movement. The amount of longitudinal bottom reinforcement is also
11 smaller than the required amount of 0.21% from NEN 6720:1995 (NEN Committee 351001
12 1995) for concrete class B45. Failure of the cross-section will result from breaking of the steel
13 reinforcement before crushing of the concrete.

14 15 ***Cracking in spans PQ and RQ***

16 The height of the cross-section in OPQRS varies from 450 mm at the side (with an
17 effective depth to the longitudinal reinforcement $d_l = 413$ mm) to 530 mm in the middle (with d_l
18 = 493 mm). The reinforcement drawings show that only 14 mm bars with a center-to-center
19 distance of 200 mm are present and continue 1.25 m past the support ($\rho_{l,bot} = 0.156\%$) and the top
20 reinforcement consists of 25 mm bars at 100 mm center-to-center ($\rho_{l,top} = 1.007\%$). The
21 transverse flexural reinforcement consists of 14 mm bars with a spacing of 200 mm for the top
22 ($\rho_{t,top} = 0.181\%$) and of 150 mm for the bottom ($\rho_{t,bot} = 0.236\%$). The transverse flexural

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

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

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

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

5

6 ***Repair actions and current situation***

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

12

13 **Aggregate interlock**

14 ***What is aggregate interlock?***

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

22

1 *The fundamental model for aggregate interlock*

2 Walraven developed a model for aggregate interlock (Walraven 1980; 1981a; Walraven
3 1981b) in which concrete is taken as a two-phase material consisting of stiff aggregate particles
4 embedded in an ideally-plastic matrix. Measurements on beams had shown that cracks do not
5 open to their final width and subsequently shear, but open and shear simultaneously. Therefore,
6 both the shear stress and normal stress have to be taken into account as essential components.
7 Assuming that the irregular faces of the crack can be deformed, both the shear stress τ and the
8 normal stress σ are functions of the crack width w and the shear displacement Δ .

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

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

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

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

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

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

12 Experimental results from push-off tests were used to determine the matrix yielding stress
13 σ_{pu} and the friction coefficient μ . The friction coefficient was found to be $\mu = 0.4$ (Walraven
14 1981b) and

$$15 \quad \sigma_{pu} = 6.39 f_c'^{0.56} \text{ (N/mm}^2\text{)} \quad (1)$$

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

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

$$22 \quad \tau = \frac{-f_c'}{30} + \left[1.8w^{-0.8} + (0.234w^{-0.707} + 0.20) f_c' \right] \Delta \quad (2)$$

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

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):

$$\tau_u = C_1 (\rho \times f_y)^{C_2} \quad (4)$$

$$C_1 = (f'_c)^{0.36} \quad (5)$$

$$C_2 = 0.09 (f'_c)^{0.46} \quad (6)$$

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.

21

1 ***Contribution of aggregate interlock to the shear capacity***

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

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

1 **Capacity of a cracked cross-section**

2 *Shear capacity based on code methods*

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

$$5 \quad V_{NEN6720} = 0.4 f_{ctd} d \times b \quad (7)$$

6 where

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

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

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

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

12

13 *Shear capacity of a section with a through crack*

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

$$16 \quad V_{agg} = \tau_u \times d \times b \quad (8)$$

17 where τ_u = the shear stress from Equation 4. Dowel action is neglected, which is a conservative
18 assumption. The reinforcement ratio is taken as half of the provided reinforcement ratio in the
19 cross-section to account for the lower bond capacity of plain reinforcement, as is commonly
20 assumed in Dutch practice. To convert Equation 4 into a design value, the result is multiplied by
21 0.85 / 1.35. The value of 0.85 takes into account the long-term effects of the concrete behavior.
22 The factor 1.35 transforms the equation for average values into an equation for characteristic
23 values. All calculations are carried out with the characteristic values of the material properties.

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

5

6 ***Maximum crack width***

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

18 *Cracks over full depth*

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

$$22 \quad V_{u_unr} = \tau \times b \times h \quad (9)$$

$$23 \quad F_{ax} = \sigma \times b \times h \quad (10)$$

- 1 where
2 τ = shear stress as given in Equation 2;
3 σ = normal stress as given in Equation 3;
4 h = height of the cross-section; and
5 $b = 1$ m.

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

14 The relation between the shear capacity V_{u_unr} and the crack width w was used to find the
15 crack width at which the shear capacity V_{u_unr} of the section with a through crack becomes
16 smaller than the shear capacity $V_{NEN6720}$ of the section without a through crack according to NEN
17 6720:1995 (NEN Committee 351001 1995). Based on the graphs that show the relation between
18 crack width and crack slip from Walraven (Walraven 1981a), it is assumed that for normal
19 strength concrete with a maximum aggregate size of 32 mm the following relation between the
20 crack width w and the shear displacement Δ can be used:

21
$$\Delta = 1.25 \times w \tag{11}$$

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

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

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

$$16 \quad F_{tc} = f_{ctk} d \times b \quad (12)$$

$$17 \quad f_{ctk} = 0.7(1.05 + 0.05 f_c') \text{ with } f_{ctk} \text{ and } f_c' \text{ in [MPa]} \quad (13)$$

$$18 \quad F_{steel} = (A_{s,bottom} + A_{s,top}) f_y \quad (14)$$

$$19 \quad F_{clamp} = F_{steel} - F_{tc} \quad (15)$$

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

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

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

3 *Axial force from restraint of deformation*

4 In a next step, the influence of the restraint of deformation on the axial tensile capacity
5 $N_{tension}$ of the cross-section is studied. The axial tensile capacity needs to be studied along with
6 the aggregate interlock capacity (shear capacity), because it can be seen in Equations 2 and 3 that
7 both the shear and axial stresses occur when a crack opens and slips. The results are summarized
8 in Table 3, in which $N_{tension}$ is the remaining axial capacity from aggregate interlock of the
9 cracked section and F_{tc} is the axial tensile force on the cross-section from Equation 12. The
10 procedure for finding $N_{tension}$ is now explained.

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

$$13 \quad w_{max, NEN 3865} = 0.8 \sigma_a \xi_2 \left(2c + \xi_3 \frac{\phi_{top}}{\rho_{top}} \right) 10^{-5} \quad (16)$$

14 where

15 c = the concrete cover;

16 $\xi_2 = 1.25$;

17 $\xi_3 = 2.5$;

18 ϕ_{top} = diameter of the top reinforcement;

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

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

$$21 \quad \sigma_a = \frac{F_{tc}}{A_{s,top}} \leq f_{yk} \quad (17)$$

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

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

$$5 \quad h_{eff} = 2.5 \left(c + \frac{\phi_{top}}{2} \right) \quad (18)$$

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

$$12 \quad V_{agg} = \tau \times h_{eff} \times b \quad (19)$$

$$13 \quad N_{agg} = \sigma \times h_{eff} \times b \quad (20)$$

14 with

15 τ = the shear stress from Equation 2;

16 σ = the axial stress from Equation 3;

17 h_{eff} from Equation 18; and

18 $b = 1$ m.

19 The horizontal equilibrium on the crack in the zone of the fictitious tension tie
20 encompasses the axial force N_{agg} (Eq. 20) from aggregate interlock for a given crack width
21 $w_{max,NEN3865}$, the tension caused by the restraint of deformation, and the clamping force provided
22 by the steel reinforcement. As a result, the remaining capacity $N_{tension}$ available to resist the

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

2 F_{top} :

3
$$F_{top} = A_{s,top} \times f_y \quad (21)$$

4
$$N_{tension} = F_{top} - N_{agg} \quad (22)$$

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

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

13 *Overview of maximum allowable crack widths*

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

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

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

3

4 **Recommendations**

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

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

12

13 **Summary and Conclusions**

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

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

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

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

3
4 **Notation List**

5 The following symbols are used in this paper:

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

7 b = width

8 c = concrete cover

9 d = effective depth of the considered cross-section

10 d_l = effective depth to the longitudinal reinforcement

11 f_c' = cube compressive strength of the concrete

12 f_{ck} = characteristic cylinder compressive strength

13 f_{ctd} = design tensile strength of the concrete

14 f_{ctk} = characteristic tensile strength of the concrete

15 f_y = yield stress

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

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

18 h = height of the cross-section

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

20 w = crack width

21 $w_{max,NEN3865}$ = maximum allowable crack width for combined flexure and tension

22 x = horizontal axis

23 y = vertical axis

24 $A_{s,bottom}$ = area of the bottom reinforcement in the cross-section

- 1 $A_{s,top}$ = area of the top reinforcement in the cross-section
- 2 C_1 = parameter in aggregate interlock formulas
- 3 C_2 = parameter in aggregate interlock formulas
- 4 F = force, not otherwise specified
- 5 F_{ax} = axial capacity based on maximum normal stress in aggregate interlock theory
- 6 F_{clamp} = resulting clamping force on the cross-section
- 7 F_{steel} = clamping force on the crack assuming yield of the reinforcement
- 8 F_{tc} = resulting tensile force
- 9 F_{top} = force in the top reinforcement assuming yield
- 10 N_{agg} = axial force from aggregate interlock
- 11 $N_{tension}$ = axial tensile capacity of the cross-section
- 12 V_{agg} = shear capacity of a section based on the ultimate aggregate interlock capacity
- 13 $V_{NEN6720}$ = design shear capacity of section without through crack according to NEN 6720:1995
- 14 (NEN Committee 351001 1995)
- 15 V_{u_unr} = shear capacity from aggregate interlock of an unreinforced cross-section
- 16 ϵ_{su} = strain at failure of the reinforcement steel
- 17 ϕ_{top} = diameter of the top reinforcement
- 18 μ = friction coefficient
- 19 ρ = reinforcement ratio
- 20 $\rho_{l,bot}$ = reinforcement ratio for the longitudinal reinforcement on the bottom of the cross-section
- 21 $\rho_{l,top}$ = reinforcement ratio for the longitudinal reinforcement on the top of the cross-section
- 22 $\rho_{t,bot}$ = reinforcement ratio for the transverse flexural reinforcement on the bottom of the cross-
- 23 section

- 1 $\rho_{t,top}$ = reinforcement ratio for the transverse flexural reinforcement on the top of the cross-
2 section
3 ρ_{top} = top reinforcement ratio (in %)
4 σ = normal stress
5 σ_a = tensile stress in the cross-section as a result of restraint of deformation
6 σ_{pu} = stress normal to the contact area
7 τ = shear stress
8 τ_u = ultimate shear stress
9 τ_{pu} = stress tangential to the contact area
10 ξ_2 = parameter in expression for allowable crack width
11 ξ_3 = parameter in expression for allowable crack width
12 Δ = shear displacement

13
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15 reinforcement after taking the tension in the cross-section into account (F_{clamp} , dashed line).

16

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3 **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

4

5

6 **Table 2.** Contribution of aggregate interlock as percentage of total shear carrying capacity at
7 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%	

8

9

10 **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

11

12