Concrete shear keys in prefabricated bridges with dry deck joints

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ABSTRACT

A prefabricated concrete deck with dry joints between deck elements has been developed to make prefabricated bridges even more competitive. This type of bridge deck has been used on single span bridges in Sweden, and is now under development for multi span bridges. This paper describes how the deck system works. Results from laboratory tests of shear keys between deck elements are also presented together with an analysis comparing the predicted capacity with the measured failure load.

Key words: Bridge, prefabrication, element, dry joints, shear keys, laboratory tests.

1. INTRODUCTION

There is always a need to widen, build or rebuild bridges. To reduce the construction time and to minimize the impact on the traffic situation, prefabricated bridges can be used. Prefabricated steel girders are rather common but prefabricated concrete deck elements are still a rare exception. In order to make prefabricated bridges even more competitive, a deck of prefabricated concrete elements with dry joints between the elements has previously been developed. This system has been used on a few single span bridges in Sweden and is now under development for multi span bridges. The aim of the R&D project is to enable the use of dry joints between elements in multi span bridges without pre-tensioning. Particular attention has been paid to the ease of manufacturing, [1][2][3].

To transfer both lateral and vertical forces through the transverse joints, and to prevent vertical displacements between the deck elements at the joints, overlapping concrete keys are used. These keys are designed as a series of overlapping male-female connections along the joints, see Figure 1 and 2. In order to ensure a good accuracy of fit in the dry joint, the elements are match cast.

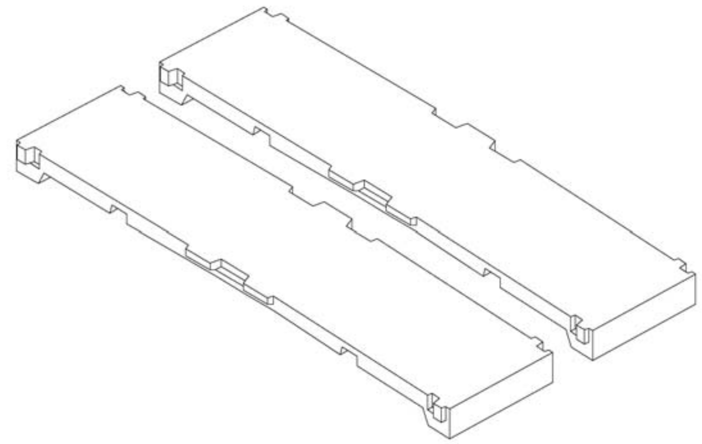

Figure 1 – 3D-sketch of two elements, illustrating the joint. *Figure 2 – Element during assembly*

The theoretical distance between the transversal reinforcement bars in the concrete deck elements and the shear studs on the steel girders is limited, and the tolerances can be demanding since the overlapping concrete keys require a longitudinal displacement of the elements at the assembling. The displacement has to be at least the horizontal depth of the overlapping concrete keys plus the tolerances in the longitudinal direction of the bridge, see Figure 3. [2]

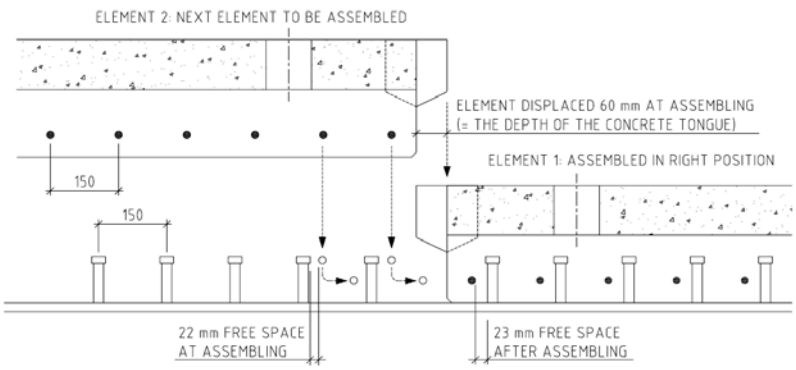

Figure 3 – Illustration of the limited tolerances. [2]

If possible, it would be preferable to use shear keys with smaller depth. However, the shear keys must be able to transfer the forces given in the design codes [4]. By using a FE-model of the bridge it can be shown that a maximum of about 40% of the traffic load acting on a single element is transferred through one of the joints. The rest of the load is transferred directly to the steel girders, or through the dry joint at the opposite side of the element. Therefore, the shear keys must be able to resist a load that is at least 40% of the design load given in the codes.

In order to find out how the shear keys transfer forces, and to be able to predict their strength and verifying the FE-model, laboratory tests have been performed.

2. Laboratory tests

Twelve static tests with three different layouts of the shear keys have been tested. The test set-up and the specimens are briefly described in the following sections.

2.1 Test set-up

The tests were focusing on pure shear capacity of the concrete keys. This means that no positive or negative effects were simulated, such as prestressing from the steel girders, or any misfit between the elements. A schematic and simplified sketch of the test set up is shown in Figure 4.

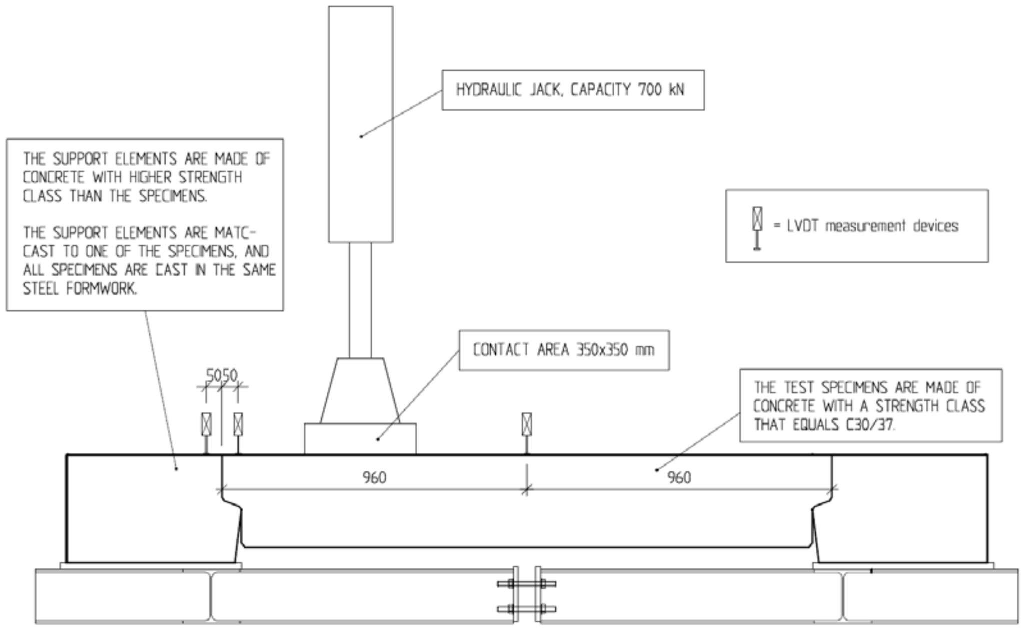

Figure 4 – Schematic and simplified sketch of the test set up.

The test specimens were placed in a test rig that consists of two concrete supports on top of a steel frame. The concrete support elements was match cast to one of the test specimens. All test specimens were made in the same steel formwork, and fitted well to the match cast supports. The supports were made in concrete with higher strength class (C40/50) than the specimens (C30/37). They were also heavily reinforced to avoid any failure in the support elements, and to make it possible to reuse them. The steel frame was used to keep the supports in the right positions and to make the demounting easier. The frame was constructed of HEA180 profiles. Figures 5 and 6 below show more detailed drawings of the support elements and the steel frame.

Before each the test was started, the bolt connections in the steel frame was tightened so that the frame could take care of the horizontal tensional forces that occurs due to the inclined contact surface in the shear keys. The bolts were tightened gently, aiming to get a remaining horizontal gap of ~0.5 mm at each shear key. The bolt connections were not used to clamp the specimen and the support elements together, giving a horizontal compressive force in the concrete.

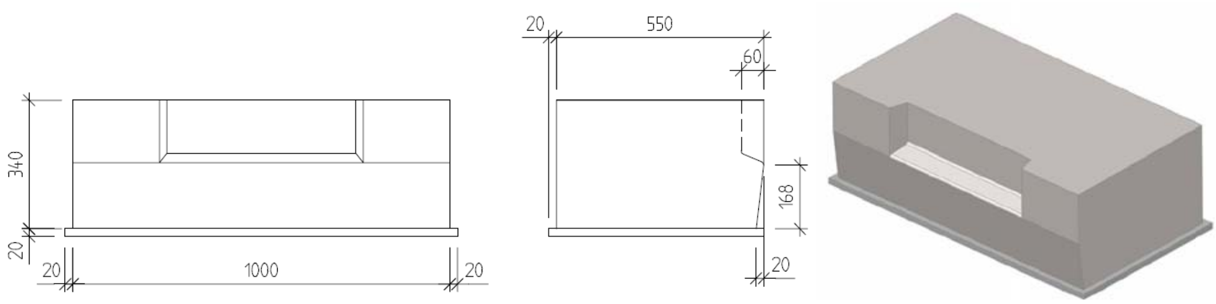

Figure 5 – Geometry drawings of support elements.

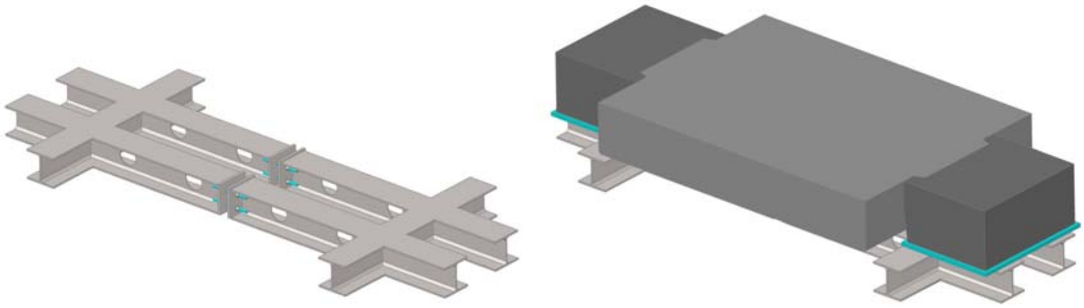

Figure 6 – Illustration of test rig.

Each specimen had two shear keys that were tested under static load. The first shear key was tested until failure. After that, the hydraulic jack was moved to the opposite side of the specimen, and the second key was tested. When the second shear key was tested an extra vertical support was used, to make sure that the specimen was levelled horizontally and that the support area was uncracked, see Figure 7.

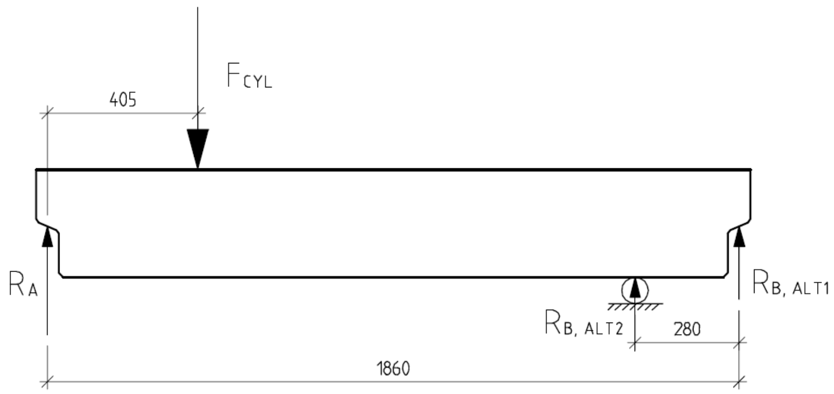

Figure 7 – Load situation with the extra support.

During the tests, data were recorded on 12 channels. Two channels were used to record the time elapsed and the load from the cylinder. The remaining ten channels were used to record deformations. Six measurement points were placed on top of the specimen. Three were placed on top of the support element and the last one was placed towards the floor measuring the reference deformation. The test set-up and the measurement devices are shown in Figure 8.

Figure 8 – Picture of test set-up.

2.2 Test specimens

The general geometry of the test specimens were 1.8 × 1.3 m, with a concrete shear key depth of 60 mm and a length of 540 mm, see Figure 9. The concrete strength in the specimens was aimed to be equal to strength class C30/37. For each specimen, six concrete cube tests were performed: 3 compressive and 3 tensile. The material test results are presented in Chapter 3.

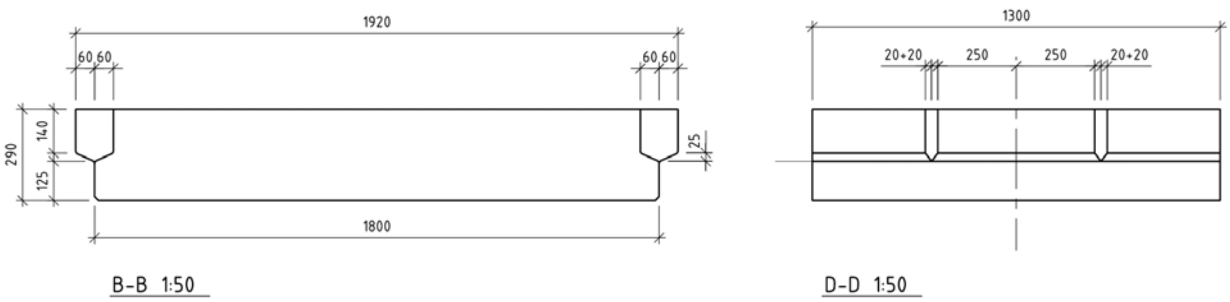

Figure 9 – General geometry of test specimens.

The first specimens (2 elements) were reinforced with exactly the same amount of reinforcement as used in deck elements in previously constructed single span bridges. In these specimens the shear keys were the same in both ends, shear key type 1. The second type of specimens (4 elements) had reduced shear key reinforcement in one of the shear keys, shear key type 2, compared to the first specimens. The other shear key, shear key type 3, was completely without reinforcement. With this design, four test results are gained for each type of shear key. Figures 10 and 11 show the reinforcement drawings of the specimens.

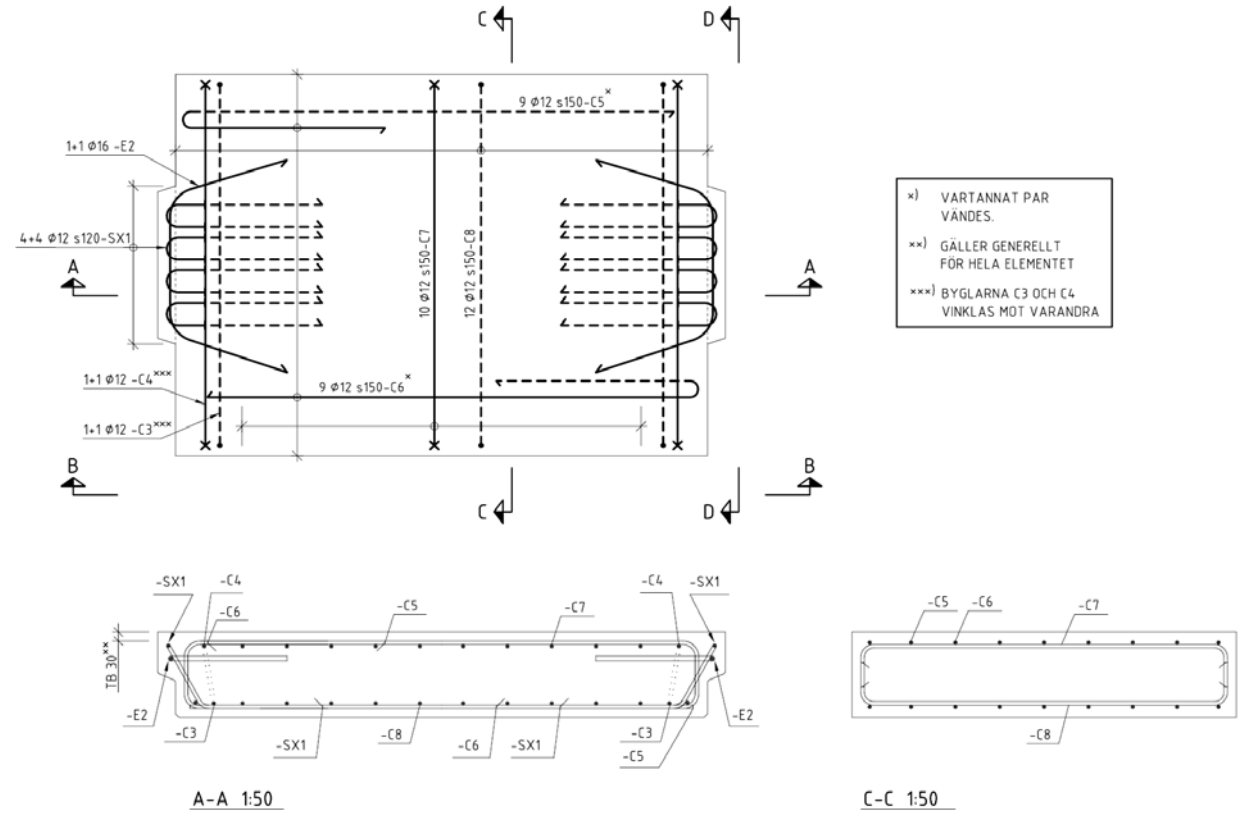

Figure 10 – Reinforcement drawing for specimens of type 1.

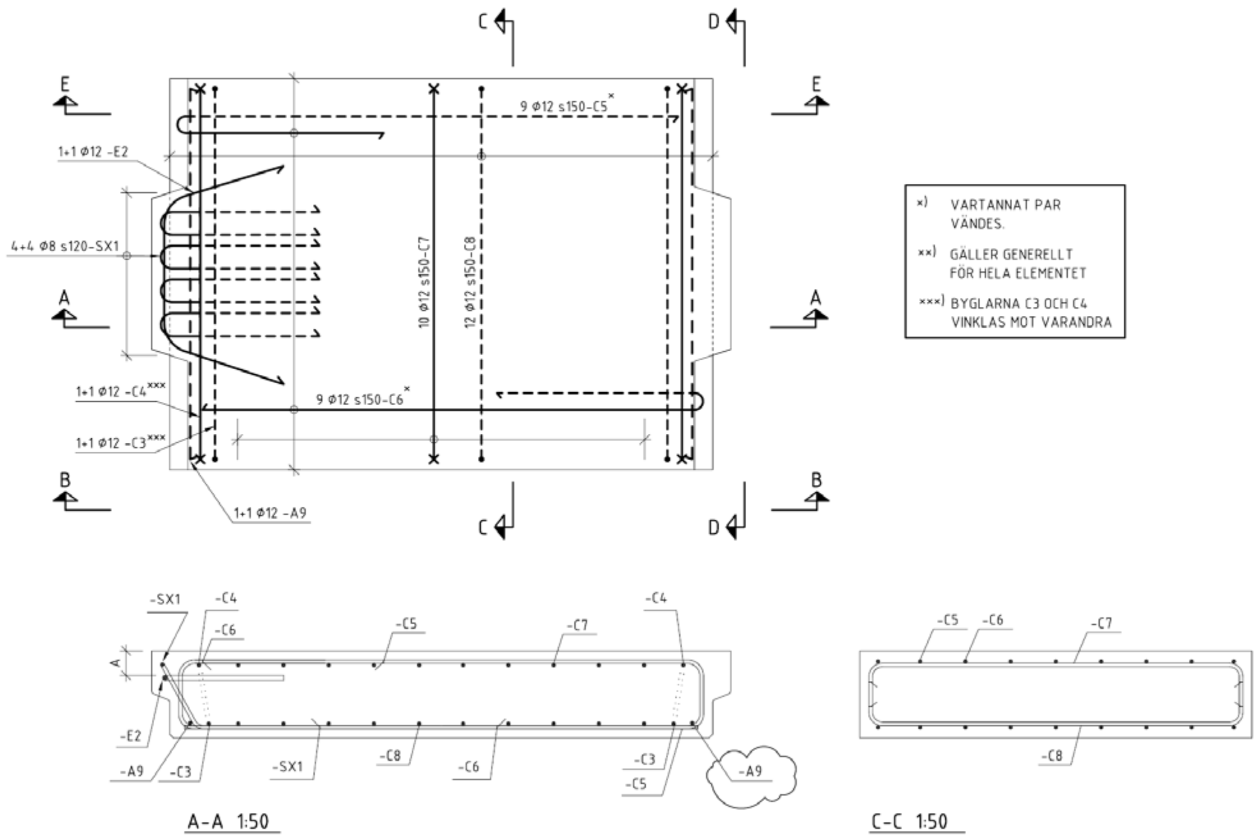

Figure 11 – Reinforcement drawing for specimens of type 2.

3. Results

3.1 Material tests

For each specimen, six cubes were cast out of the same concrete mix. Three of the cubes were used to determine the compressive strength and the other three were used to determine the tensile strength. The test cubes had the dimensions of 150×150×150 mm. The mean values for each specimen are presented in Table 1 below.

Table 1 – Concrete parameters.

Cast date	Test date	Age [days]	δ [kg/m³]	P_c [kN]	f_c [MPa]	P_{ct} [kN]	f_{ct} [MPa]	Specimen type
2010-03-15	2010-06-16	93	2334	1045	46.1	118	2.6	1
2010-03-16	2010-06-11	87	2345	1132	49.7	123	2.8	1
2010-03-18	2010-06-02	76	2372	1082	47.6	103	2.3	2
2010-04-07	2010-06-08	62	2330	967	42.6	94	2.1	2
2010-04-08	2010-06-11	64	2358	1009	44.5	115	2.6	2
2010-04-12	2010-05-31	49	2371	970	42.9	99	2.3	2

P_c = failure load, compressive test
P_{ct} = failure load, splitting test
f_c = compressive strength
f_{ct} = splitting tensile strength

3.2 Shear key tests

All tests were deformation controlled, with a stroke of 0.02 mm/s. Two different kinds of failures were observed when the reinforced shear keys were tested. Firstly, five of eight reinforced shear keys failed by cracks that activated the reinforcement, giving a ductile behaviour – failure type 1. The shear keys remained as one piece, but with some concrete crushing in the lower parts. Three specimen failed by cracks that were developed outside the reinforcement, resulting in a failure that separated the shear key from the rest of the specimen – failure type 2. This type of failure occurred under lower loads than the previously described failure.

Shear key type 1 – Ø12 reinforced (4 tests)
Two of four shear keys of type 1 resulted in failure type 1. The load-deformation curves from these two tests are shown in Figure 12 with solid lines, together with some photos of the failed shear keys, Figure 13.

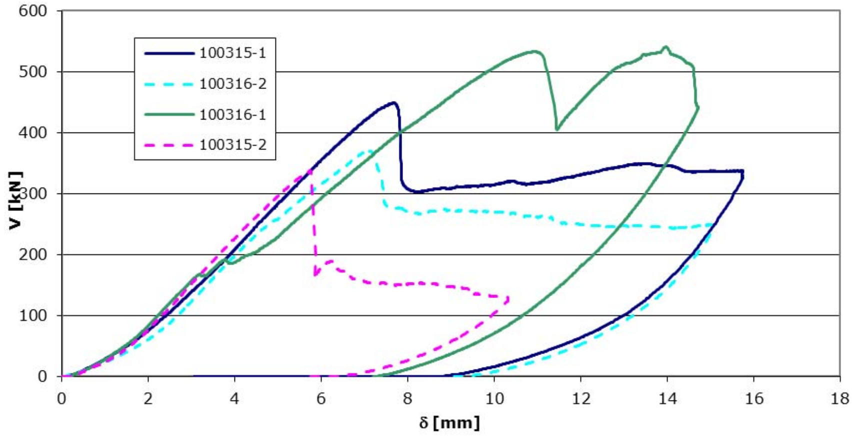

Figure 12 – Load-deformation curve for shear key type 1, failure type 1 (solid) and 2 (dashed).

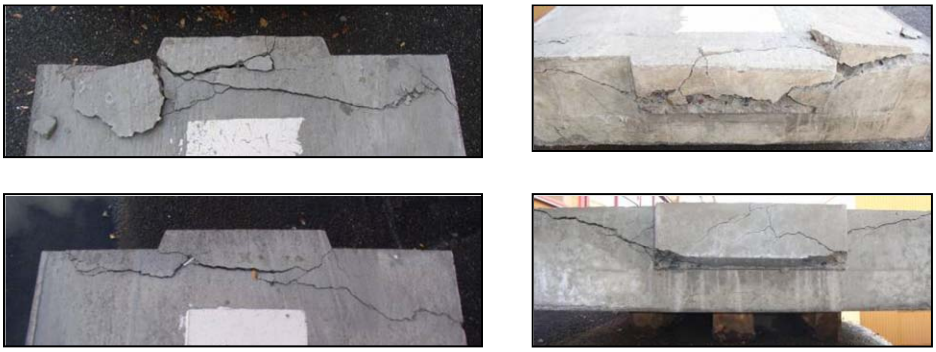

Figure 13 – Photos of shear key type 1 with a failure activating the reinforcement.

The last two specimens failed by cracks that developed outside the reinforcement, resulting in a failure that separated the shear key from the rest of the specimen. The load-time curves from these two tests are shown with dashed lines in Figure 12, together with some photos of the failed shear keys, see Figure 14.

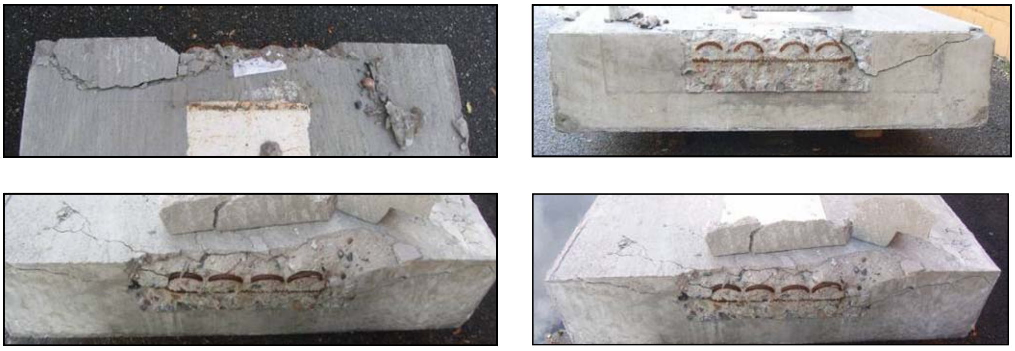

Figure 14 – Photos of shear key type 1 after failure in the concrete covering layer.

Shear key type 2 – Ø8 reinforced (4 tests)
The load-time curves from the tests are shown below together with some photos of the failed shear keys, see Figures 15 and 16.

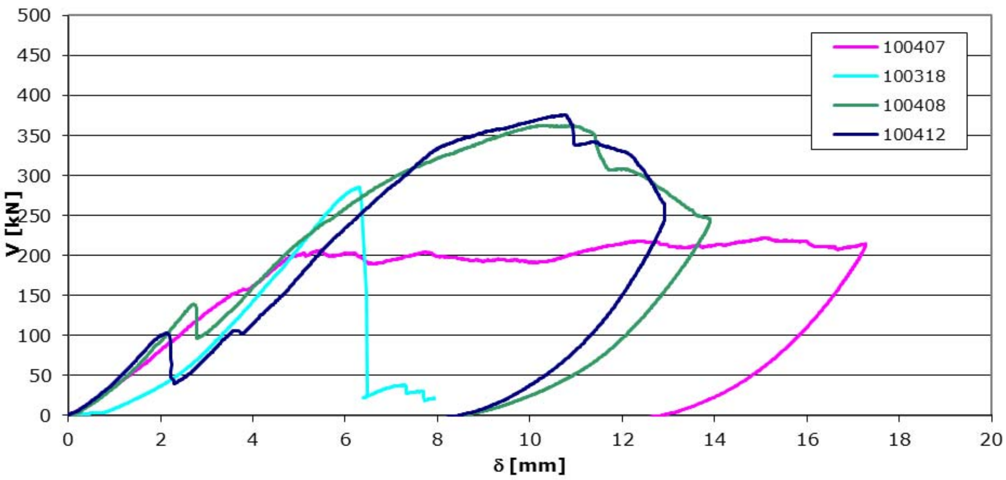

Figure 15 – Load-time curve for shear key type 2.

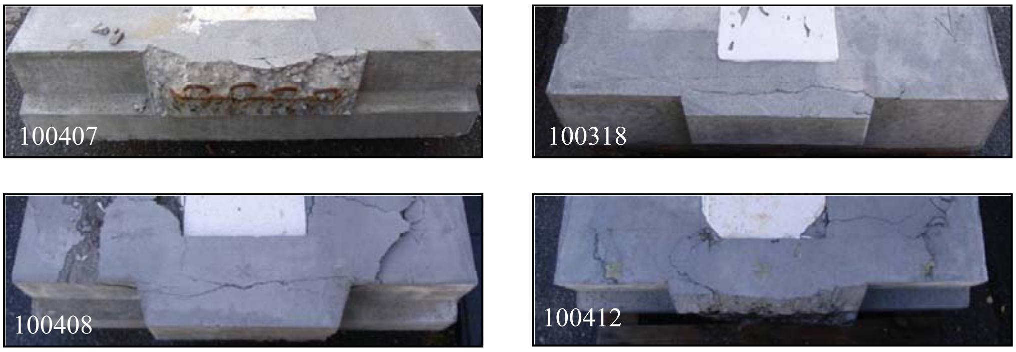

Figure 16 – Photos of shear key type 2 after failure.

In one aspect, the results from these tests reminds of the tests of shear key type 1 – Ø12 reinforced, since three shear keys remains rather unaffected after cracking, and one shear key fail outside the reinforcement. The latter shows a very plastic behaviour before it finally fails in the concrete cover layer.

Shear key type 3 – unreinforced (4 tests)
The load-time curves from the tests are shown below together with some photos of the failed shear keys, see Figures 17 and 18.

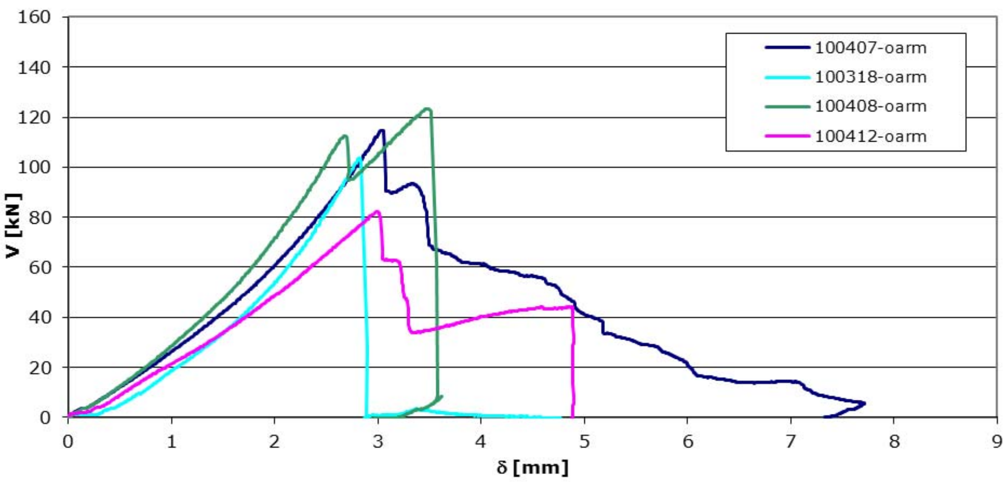

Figure 17 – Load-time curve for shear keys of type 3

Figure 18 – Photos of shear-keys after failure.

4. Analysis

The strength of the shear keys has been estimated by using four different design models:

1. classical beam linear elastic analysis – shear key type 3 with failure type 2
2. Eurocode 2 – shear key type 1 and 2 with failure type 2 [6]
3. Eurocode 2 – shear key 1 and 2 with failure type 1 [6]
4. force equilibrium model

The material parameters from the test of concrete cubes are used to calculate the shear capacity for each shear key. The results from the design models are presented in Table 2.

1. Shear resistance for shear key type 3 with failure type 2

The shear resistance for failure type 2 in shear key type 3 without reinforcement can be estimated as, according to classic beam analysis and assuming that the shear strength is half the tensile strength [7],

$$V_{Rd,c} = \frac{2}{3} b_w h f_{shear} \approx \frac{1}{3} b_w h f_{ct} \tag{1}$$

where

b_w = 540 mm; the smallest width of the cross-section within the effective height

h = 165 mm; height of shear key

f_{ct} = is the splitting tensile strength of the concrete, see Table 1

2. Shear resistance for shear key type 1 and 2 with failure type 2 [5],

The shear resistance for failure type 2 can also be estimated by using formulas from [5],

$$V_c = b_w d f_v \tag{2}$$

$$f_v = 0.3 \cdot \xi \cdot (1 + 50\rho) f_{ct} \tag{3}$$

$$\rho = A_{s0} / (b_w \cdot d) \leq 0{,}02 \tag{4}$$

$\xi = 1.4$ when $d \leq 0.2$ m

d = 140 mm

b_w = 540 mm

b_w the smallest width of the cross-section within the effective height

d effective height

f_v shear strength of the concrete

f_{ct} tensile strength of the concrete

A_{s0} the smallest amount of bending reinforcement in the tensile part of the studied cross-section. This is set to 0, since there is no bending reinforcement in the shear key, only shear reinforcement.

3. Shear resistance for shear key 1 and 2 with failure type 1 [6],
This approach has been used on at least two bridges in Sweden, a bridge over Rokån and a bridge in Norrfors.

According to Eurocode 2 [6] the shear resistance for a section with inclined shear reinforcement is

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha \tag{5}$$

When shear reinforcement is used locally, with inclined rebars in one line (the –SX rebars), then the equation above can be simplified to

$$V_{Rd,s} = A_{sw} f_{ywd} \sin\alpha \tag{6}$$

where

A_{sw} = is the area of the shear reinforcement

f_{yw} = 500 MPa, is the yield strength of the shear reinforcement

θ = is the angle of the shear crack (~45° observed in the test)

α = 60° is the inclination of the shear reinforcement

4. Force equilibrium model.
This model has been suggested, by Dr. Bo Westerberg (KTH, Stockholm), in order to describe the load carrying capacity in more detail. It is a force equilibrium model that involves both the reinforcement and compressive struts in the concrete, see Figure 19.

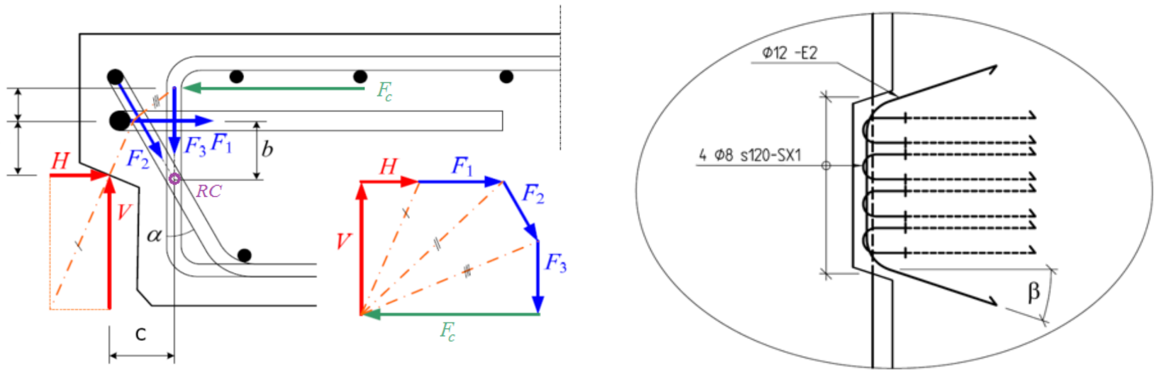

Figure 19 – Illustration of force equilibrium model and the notations.

The horizontal force *H* is believed to have great influence on the load carrying capacity. Without a compressive horizontal force, there is a risk for shear failure of the concrete cover at the edge of the shear key. The load carrying capacity is hard to predict in such a scenario, but it cannot be more than the shear strength of the concrete.

In the laboratory tests, the size of the force *H* is dependent on the shape of the supports, the rigidity of the test-rig etc. In a real bridge, this force will vary along the bridge and will depend on the global load situation as well as the local load situation.

Equilibrium equations:

$$\Sigma F \uparrow: \quad V - F_2 \cos\alpha - F_3 = 0 \tag{7}$$

$$\Sigma F \rightarrow: \quad H - F_c + F_2 \sin\alpha + F_1 = 0 \tag{8}$$

$$\Sigma M_{RC}: \quad V \cdot c + F_1 b + H(b-a) - F_c(b+z) = 0 \tag{9}$$

The load carrying capacity of the shear key can be estimated by using the maximum capacity of each rebar.

$$F_1 \le f_y A_{s1} \cos\beta \tag{10}$$

$$F_2 \le f_y A_{s2} \tag{11}$$

$$F_3 \le f_y A_{s3} \tag{12}$$

giving

Ø$_1$ = 12 mm

$$F_{1,\max} = 500 \cdot 2 \frac{\pi \cdot 16^2}{4} \cdot \cos 18.5° = 191 \text{ kN}$$

$$F_{2,\max} = 500 \cdot 8 \frac{\pi \cdot 12^2}{4} = 452 \text{ kN}$$

$$F_{3,\max} = 500 \cdot 3 \frac{\pi \cdot 12^2}{4} = 170 \text{ kN}$$

Ø$_2$ = 8 mm

$$F_{1,\max} = 500 \cdot 2 \frac{\pi \cdot 12^2}{4} \cdot \cos 18{,}5° = 107 \text{ kN}$$

$$F_{2,\max} = 500 \cdot 8 \frac{\pi \cdot 8^2}{4} = 201 \text{ kN}$$

$$F_{3,\max} = 500 \cdot 3 \frac{\pi \cdot 12^2}{4} = 170 \text{ kN}$$

As a first assumption the horizontal force, *H*, is set equal to zero. Then we assume that we are utilizing the shear reinforcement up to 100%. This gives the following result by Equation (7).

Ø$_1$ = 12 mm

$$V_{\max} = 452 \cos 30° + 170 = 561 \text{ kN}$$

Ø$_2$ = 8 mm

$$V_{\max} = 201 \cos 30° + 170 = 344 \text{ kN}$$

The moment equilibrium Equation (9) gives:

Ø$_1$ = 12 mm

$$F_c = \frac{561 \cdot 66 + 191 \cdot 65}{65 + 20} = 582 \text{ kN}$$

Ø$_2$ = 8 mm

$$F_c = \frac{344 \cdot 66 + 107 \cdot 65}{65 + 20} = 349 \text{ kN}$$

Assuming that the compressive strut in the concrete is developed over a height of 30 mm and the width of 540 mm, the compressive stress in the concrete can be calculated as:

Ø$_1$ = 12 mm

$\sigma_{Fc} = F_c/(w \cdot h)$

$\sigma_{Fc} = 582/(540 \cdot 30) = 35.9$ MPa

Ø$_2$ = 8 mm

$\sigma_{Fc} = F_c/(w \cdot h)$

$\sigma_{Fc} = 353/(540 \cdot 30) = 21.8$ MPa

These compressive stresses are below the compressive strengths that have been measured, and failures caused by concrete crushing could not be observed in the tests. The assumed distribution of forces in the rebars would be a possible solution according to this load model, resulting in yielding in the shear reinforcement. Anyhow, this is only one possible solution for this model, based on theoretical positions of the rebars. This load model should be calibrated to the test results, as should the influence of the horizontal force *H*. For example, frictional forces between the concrete surfaces will influence the result.

5. Test results vs. calculation models

Table 2 – Test results compared to results from calculation models.

Cast date	Test results V_{max} [kN]			Model 1 V_{max} [kN]				Model 2 V_{max} [kN]				Model 3 V_{max} [kN]				Model 4 V_{max} [kN]			
2010-	Ø12	Ø8	-	Ø12	Ø8	-	η*	Ø12	Ø8	-	η*	Ø12	Ø8	-	η*	Ø12	Ø8	-	η*
03-15	449	-	-	80	-	-	*5.61*	*84*	-	-	*5.36*	392	-	-	*1.15*	561	-	-	*0.80*
03-15	337	-	-	80	-	-	*4.21*	*84*	-	-	*4.03*	392	-	-	*0.86*	561	-	-	*0.60*
03-16	532	-	-	83	-	-	*6.41*	*88*	-	-	*6.07*	392	-	-	*1.36*	561	-	-	*0.95*
03-16	370	-	-	86	-	-	*4.30*	*88*	-	-	*4.22*	392	-	-	*0.94*	561	-	-	*0.66*
03-18	-	285	-	-	68	-	*4.19*	-	*73*	-	*3.88*	-	174	-	*1.64*	-	344	-	*0.83*
03-18	-	-	104	-	-	68	*1.53*	-	-	73	*1.42*	-	-	0	-	-	-	0	-
04-07	-	222	-	-	63	-	*3.52*	-	67	-	*3.30*	-	174	-	*1.28*	-	344	-	*0.65*
04-07	-	-	114	-	-	63	*1.81*	-	-	67	*1.70*	-	-	0	-	-	-	0	-
04-08	-	363	-	-	77	-	*4.71*	-	82	-	*4.42*	-	174	-	*2.09*	-	344	-	*1.06*
04-08	-	-	123	-	-	77	*1.60*	-	-	82	*1.50*	-	-	0	-	-	-	0	-
04-12	-	376	-	-	68	-	*5.53*	-	71	-	*5.30*	-	174	-	*2.16*	-	344	-	*1.09*
04-12	-	-	82	-	-	68	*1.21*	-	-	71	*1.16*	-	-	0	-	-	-	0	-

* η = test result divided by the predicted value for the given calculation model.

According to the result presented in Table 2, calculation model 1 and 2 can be useful to estimate the strength for a shear key without reinforcement. The design values are on the safe side with a safety factor from 1.16 – 1.81. Model 3 gives results that are on safe side except for the failures in the concrete cover for test specimen type 1. With the assumptions made, design model 4 is the same as model 3, except the fact that model 4 makes the vertical reinforcement bars in the slab active. The result is often on the unsafe side, which could indicate that the vertical rebars does not influence the load carrying capacity as much as assumed in the calculations. This model needs to be studied more detailed, calibrated to the test results and maybe modified.

One thing that can be noted is that the shear keys that fail in the concrete covering layer still transfer forces that are far higher than the capacity of the concrete itself. Therefore, the reinforcement must have been activated, and should be included in the design formula in one way or another.

6. Conclusions

The results from the tests have a considerable scatter. Still some interesting points can be noted. Firstly, the tests show that unreinforced concrete can not transfer the design shear forces, caused by the vehicle models in Eurocode, from one element to another. This was an expected result, in line with the result from the calculations. However, in the reality we believe that the shear keys can transfer a higher load since the surrounding elements will deflect together with the loaded element, which probably gives longitudinal compressive forces which would counteract the tensile stresses that occurs due to the shear forces.

Secondly, the load carrying capacity of the previously used shear keys seems to be larger than necessary, especially if we can avoid a failure that is developed by a crack growing through the concrete covering. The shear keys with less amount of reinforcement (Ø8mm) are still strong enough to carry the load. However, we suggest some changes in the shear key reinforcement since we see some potential improvement. Two new reinforcement layouts are presented in Figure 18. In both cases the concrete cover layer has been reduced. In the left case, additional reinforcement have been added (-E8), and in the right case the SX-rebars have been replaced by EX-rebars. The geometry of the EX- and SX- rebars can be seen in Figure 10 and 20. By using stainless steel in some of the rebars in the shear keys, it would be possible to decrease the thickness of the concrete covering layer, and hopefully avoid a failure in the covering layer.

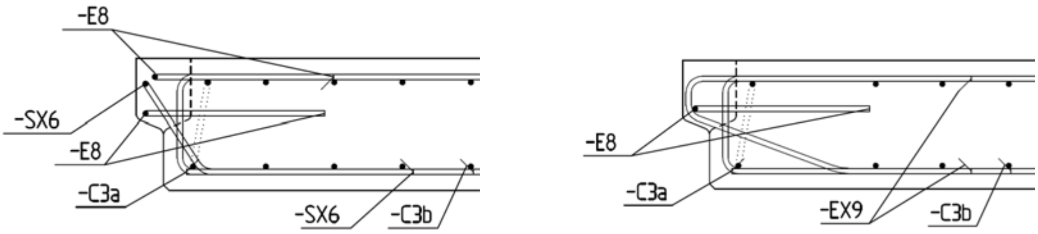

Figure 20 – Illustration of two suggested reinforcement layouts.

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