Effects of Surface Reinforcement on Bearing Capacity of Concrete with Anchor Bolts

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ABSTRACT
66 tests of the influence of surface reinforcement on the tensile load capacity of cast-in fasteners have been carried through. In the tests threaded rods ∅30 mm with a ∅45 mm nut at the end were cast-in centrically in concrete slabs (1.2 m × 1.2 m × 0.3 m up to 2.2 m × 2.2 m × 0.6 m). The amount of surface reinforcement was varied from 0% up to about 1.2%. There is a considerable increase in the load capacity with surface reinforcement present. The increase depends on the geometry and the amount and placement of the reinforcement.

Key words: fasteners, load capacity, test, reinforcement

1. INTRODUCTION

1.1 General

Fasteners of different kinds are often used to anchor loads in concrete structures. Cast-in-place reinforcement bars have been used ever since reinforced concrete was introduced around 1900. Post-installed mechanical anchors started to be used in the 1960ies with the advance of drilling machines; and bonded anchors were developed during the 1970ies. An overview of the technology is given in [1] and [2]. Typical fasteners used in nuclear applications are cast-in-place reinforcement bars and headed studs. Post-installed mechanical anchors have also been installed during maintenance of the plants. Today mostly fasteners with under-cut anchors are used for new installations in existing structures.

Pipes in nuclear power plants are often fastened to plates that are anchored in concrete walls and slabs by anchor bolts. Design rules are often conservative and based on testing on bonded and/or expansion anchors fastened in un-reinforced concrete. A need for more appropriate rules has been called for in connection with a planned increase of the effect in Swedish nuclear power plants. This is studied within a project named PULS (Power Update under Licensed Safety, OKG 3, Oskarshamn). By testing some actual types of fastenings in reinforced concrete slabs it should be possible to develop more adequate and, hopefully, less conservative design models.

The aim and scope of this paper is to present results from the tests and analyses in order to form a background for revised design recommendations for anchor bolts compared to the methods proposed in Germany, [3] and [4].

2 CODES AND STANDARDS

The American Society of Mechanical Engineers, ASME, published the first rules for pressure vessels in 1911 and the American Nuclear Society, ANS, has published codes for nuclear plants since 1957, see e.g. [5]. Present codes for loads and design of pipes and fastenings in nuclear power plants are given in [6].

The American Concrete Institute, ACI, published the first code for anchor bolts for nuclear applications in 1985 and it has since been developed successively, see [7]. The anchor bolt code is now also included as Appendix D in the general concrete code, [8].

Fastenings to structural concrete and masonry has since 1987 been the subject of a Special Activities Group of the International Federation for Structural Concrete (fib - fédération internationale du béton) and its predecessor the European Committee for Concrete (CEB - Comité Européen du Béton). The group has published a state of the art report and a Design Guide, [9]. A revised version of the design guide is presently under preparation, [10]. An overview of codes for fasteners is given by [11] and [3].

Round Robin tests and analysis of anchor bolts have been carried out in Sweden and internationally as well as cyclic and fatigue tests of anchor bolts and long-time loading of bonded anchors, see [12], [13], [14], [15] and [16].

2.1 Failures

The capacity of a fastener is governed by its geometry and materials and the properties of the concrete it is fastened in. Several failure modes are possible: failures associated with tensile loading (pullout, pull-through, combined pullout and concrete cone, concrete cone, splitting, steel) and failures associated with shear loading (steel, edge breakout, pry out, pullout). See [10] for a broader description of the different types of failures.

The tests presented in this paper are anchor-bolts subjected to tensile loading.

The resistance in the case of a concrete cone failure, P_c [N], see Figure 1, can, based on tests, see e.g. [2], and Fracture Mechanics size effect relations, be written as

$$P_c = k f_{c,cyl}^{0.5} h_{ef}^{1.5} \tag{1}$$

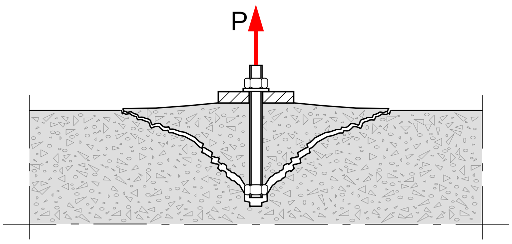

Figure 1 – Concrete cone failure.

where
- k = an empirical constant (around 15 $N^{0.5}/mm^{0.5}$ for undercut and headed anchors)
- $f_{c,cyl}$ = concrete cylinder strength [MPa]
- h_{ef} = effective depth of the anchor [mm]

If several bolts are placed close to one another the capacity is reduced according to the so called CC method (Concrete Capacity) by multiplication with ψ-coefficients. The method was presented by [17] and is also given in [2] and [8].

The capacity for a cast-in-place rebar or a bonded anchor is often governed by a combination of a pull-out and a cone failure, P_p [N], which can be determined from

$$P_p = \tau_R \pi d h_{ef} \tag{2}$$

where
- τ_R = bond resistance for cracked or un-cracked concrete [MPa]
- d = diameter of bar or drilled hole [mm]

Splitting failure due to anchor loading is usually not verified if adequate surface reinforcement is provided and anchors suitable for application in cracked concrete are used.

Dynamic properties of fasteners have recently been tested by [18], [19] and [11]. The tests indicate that dynamic loads can give a higher resistance than a static load.

It has recently been shown that the assumptions that concrete failures are much more brittle than steel failures are wrong for normal anchor depths, where the yielding of the steel only can provide a very limited deformation, [11].

Fatigue tests indicate that failure occurs when the deformation due to cyclic loading grows larger than the corresponding deformation at a static test. Empirical formula for this is given by [11].

2.3 Comparison between different codes

There are different Load Levels: A, B, C and D and Plant Conditions: PC-1 to PC-5 according to Swedish code [20] and the US codes [6] and [5] with different partial safety factors. According to [5] the spectrum of normal operations and events shall be identified in accordance with their best-estimate frequency of occurrence and divided into five categories according to Table 1.

Table 1 – Plant Condition, PC, frequency/year (F), corresponding Swedish class (Händelse, H) and Load Level. [5], [6] and [20].

Plant Cond.	Best Estimate Frequency of Occurrence (F) per Reactor Year	SSMFS 2008:17 (2008)	Load Level
PC 1	Normal operations	H 1	A
PC 2	$10^{-1} \leq F$	H 2	B
PC 3	$10^{-2} \leq F < 10^{-1}$	H 2	B
PC 4	$10^{-4} \leq F < 10^{-2}$	H 3	C
PC 5	$10^{-6} \leq F < 10^{-4}$	H 4 H 5	D

2.4 Potential for Anchor Bolts

In [3] the German Code [21] is used to calculate the characteristic capacity for a cone failure in cracked concrete, $N_{Rk,c}$. The factor k in (1) is given the value k = 8.9 for Load Level A in Table 1 above.

$$P_{Rk,c} = 8.9\, f_{c,cyl}^{0.5} h_{ef}^{1.5} \tag{3}$$

For extreme loads corresponding to Load Level D, a further reduction is made with the factor $\alpha_{ductile}$ = 0.75 for ductile failures alternatively by a factor $\alpha_{brittle}$ = 0.54 for brittle failures. The brittle failure thus gives a capacity which is 0.54/0.75 = 0.72 smaller than the ductile failure.

3 TESTS

In total 66 tests have been carried through for testing the influence of surface reinforcement on the load bearing capacity of cast-in headed bars. In all tests threaded rods with diameter ∅30 mm and a ∅45 mm nut at the end were cast-in centrically in a concrete slab. The width, length and thickness of the concrete slabs were varied from 1.2 m × 1.2 m × 0.3 m up to 2.2 m × 2.2 m

× 0.6 m. In the slabs the amount of top reinforcement was varied from 0% up to about 1.2% (∅16 #100). About half of the tests were cracked centrically through the cast-in rod; the width of the cracks was about 0.5 mm, see Tables 2 and 3.

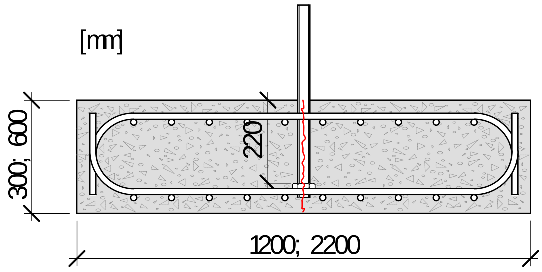

Figure 3 – Typical test slab with cast-in threaded rod and nut, top and bottom reinforcement and initial crack going through the location of the rod.

Table 2 – Summary of test setup: test type, size, concrete cover, amount of top reinforcement, and initial crack width.

Test type.	*L* [m]	H [m]	*c* [mm]	Top reinf.	Crack [mm]
1	1.2	0.3	30	∅16 #100	0.5
2	1.2	0.3	30		0.5
3	1.2	0.3	30	∅16 #100	0
4	1.2	0.3	30		0
10	1.2	0.3	30	∅12 #300	0
11	1.2	0.3	30	∅12 #300	0.5
12	1.2	0.3	30	∅16 #150	0
13	1.2	0.3	30	∅16 #100	0.5
14	1.2	0.6	30	∅12 #300	0
15	1.2	0.3	50	∅16 #100	0.5
16	2.2	0.3	30	∅12 #150	0
17a	2.2	0.3	30	∅12 #400	0
17b	2.2	0.3	30	∅12 #500	0.5
18	2.2	0.6	30	∅12 #150	0

3.1 Material data

The test specimens were manufactured by Bröderna Hedmans Cementgjuteri in Älvsbyn, Sweden. The threaded rods were of quality 8.8 (f_{ud} = 800 MPa, f_{yk} = 640 MPa). The reinforcement was of quality B500B (f_{yk} = 500 MPa) and the concrete, of quality C25/30, was mixed using the following recipe: gravel 8-16 mm 760 kg/m³, sand 0-8 mm 1050 kg/m³, cement 350 kg/m³, w/c = 0.57.

3.2 Test setup

The test setup is illustrated in Figure 4. The support was for all but the first 12 slabs a stiff ring with an inner diameter of 1.1 m or 2.1 m placed on a thin gypsum layer. The first four slabs were supported at the four corners while the next eight slabs where supported on a ring without or with a thin rubber layer.

Figure 4 – Test set-up: support ring with radius 0.55 m (left) and with radius 1.1 m (right).

Table 3 – Test data: size, amount of top reinforcement, crack width, failure load and deformation.

No.	W [m]	L [m]	H [m]	Top reinf.	Crack [mm]	P^* [kN]	Def. [mm]
1	1.2	1.2	0.3	–	0	186.5	0.36
2	1.2	1.2	0.3	∅16 #100	0	285.3	1.85
3	1.2	1.2	0.3	–	0.5	146.7	4.61
4	1.2	1.2	0.3	∅16 #100	0.5	246.9	2.79
5	1.2	1.2	0.3	–	0	215.5	30.42
6	1.2	1.2	0.3	∅16 #100	0	308.2	1.35
7	1.2	1.2	0.3	–	0.5	165.9	1.94
8	1.2	1.2	0.3	∅16 #100	0.5	240.3	1.58
9	1.2	1.2	0.3	–	0	176.8	0.49
10	1.2	1.2	0.3	∅16 #100	0	267.5	1.74
11	1.2	1.2	0.3	–	0.5	121.7	22.01
12	1.2	1.2	0.3	∅16 #100	0.5	276.3	2.23
13	1.2	1.2	0.3	∅16 #100	0.5	231.7	2.21
14	1.2	1.2	0.3	∅16 #100	0.5	266.8	1.88
15	1.2	1.2	0.3	∅16 #100	0.5	257.2	1.90
16	1.2	1.2	0.3	∅16 #100	0	281.7	5.70
17	1.2	1.2	0.3	∅16 #100	0	329.1	1.78
18	1.2	1.2	0.3	∅16 #100	0	341.4	1.96
19	1.2	1.2	0.3	∅16 #150	0	321.5	9.79
20	1.2	1.2	0.3	∅16 #150	0	321.9	8.73
21	1.2	1.2	0.3	∅16 #150	0	296.5	3.92
22	1.2	1.2	0.3	∅16 #150	0	325.5	6.99
23	1.2	1.2	0.3	∅16 #150	0	332.2	7.92
24	1.2	1.2	0.3	∅16 #150	0	313.5	8.31
25	1.2	1.2	0.3	∅16 #150	0.5	294.3	4.76
26	1.2	1.2	0.3	∅16 #150	0.5	283.8	4.97
27	1.2	1.2	0.3	∅16 #150	0.5	305.4	4.54
28	1.2	1.2	0.3	∅16 #150	0.5	318.8	8.38
29	1.2	1.2	0.3	∅16 #150	0.5	311.7	9.32
30	1.2	1.2	0.3	∅16 #150	0.5	305.1	7.95
31	1.2	1.2	0.3	∅16 #100	0.5	316.3	8.99
32	1.2	1.2	0.3	∅16 #100	0.5	302.9	9.17
33	1.2	1.2	0.3	∅16 #100	0.5	321.7	3.74
34	1.2	1.2	0.3	∅16 #100	0.5	296.4	2.99
35	1.2	1.2	0.3	∅16 #100	0.5	279.1	2.89
36	1.2	1.2	0.3	∅16 #100	0.5	292.9	11.21
37	1.2	1.2	0.3	∅12 #300	0.5	278.7	7.17
38	1.2	1.2	0.3	∅12 #300	0.5	288.9	8.25
39	1.2	1.2	0.3	∅12 #300	0.5	277.9	8.38
40	1.2	1.2	0.3	∅12 #300	0.5	293.8	10.30
41	1.2	1.2	0.3	∅12 #300	0.5	314.0	11.65
42	1.2	1.2	0.3	∅12 #300	0.5	297.5	9.37
43	1.2	1.2	0.3	∅12 #300	0	292.4	9.83
44	1.2	1.2	0.3	∅12 #300	0	287.3	7.85
45	1.2	1.2	0.3	∅12 #300	0	255.4	8.56
46	1.2	1.2	0.3	∅12 #300	0	291.7	9.62
47	1.2	1.2	0.3	∅12 #300	0	249.2	8.72
48	1.2	1.2	0.3	∅12 #300	0	306.4	10.76
49	2.2	2.2	0.3	∅12 #150	0	256.8	8.07
50	2.2	2.2	0.3	∅12 #150	0	262.2	7.62
51	2.2	2.2	0.3	∅12 #150	0	267.2	8.36
52	1.2	1.2	0.6	∅12 #300	0	330.8	8.85
53	1.2	1.2	0.6	∅12 #300	0	365.1	10.24
54	1.2	1.2	0.6	∅12 #300	0	361.9	8.89
55	1.2	1.2	0.6	∅12 #300	0	370.1	10.23
56	1.2	1.2	0.6	∅12 #300	0	344.2	9.83
57	1.2	1.2	0.6	∅12 #300	0	371.0	9.61
58	2.2	2.2	0.3	∅12 #400	0	247.5	9.53
59	2.2	2.2	0.3	∅12 #400	0	241.3	9.78
60	2.2	2.2	0.3	∅12 #400	0	233.9	7.00
61	2.2	2.2	0.3	∅12 #500	0.5	204.2	4.68
62	2.2	2.2	0.3	∅12 #500	0.5	235.7	7.47
63	2.2	2.2	0.3	∅12 #500	0.5	212.5	8.11
64	2.2	2.2	0.6	∅12 #150	0	319.5	7.12
65	2.2	2.2	0.6	∅12 #150	0	333.8	10.24
66	2.2	2.2	0.6	∅12 #150	0.9	261.4	10.74

The compressive and tensile (splitting) strength of the concrete were tested for every slab in order to correct for their age compared to the 28 days strengths.

4 RESULTS

The results of the tests are shown in Figure 5-Figure 7. In Figure 5 the results from the 1.2 m × 1.2 m × 0.3 m specimen without a crack are presented for four different amounts of reinforcement: Ø16 #100 in the bottom, Ø16 #100 in the top and the bottom, Ø16 #150 in the top and bottom, and Ø12 #300 in the top and the bottom.

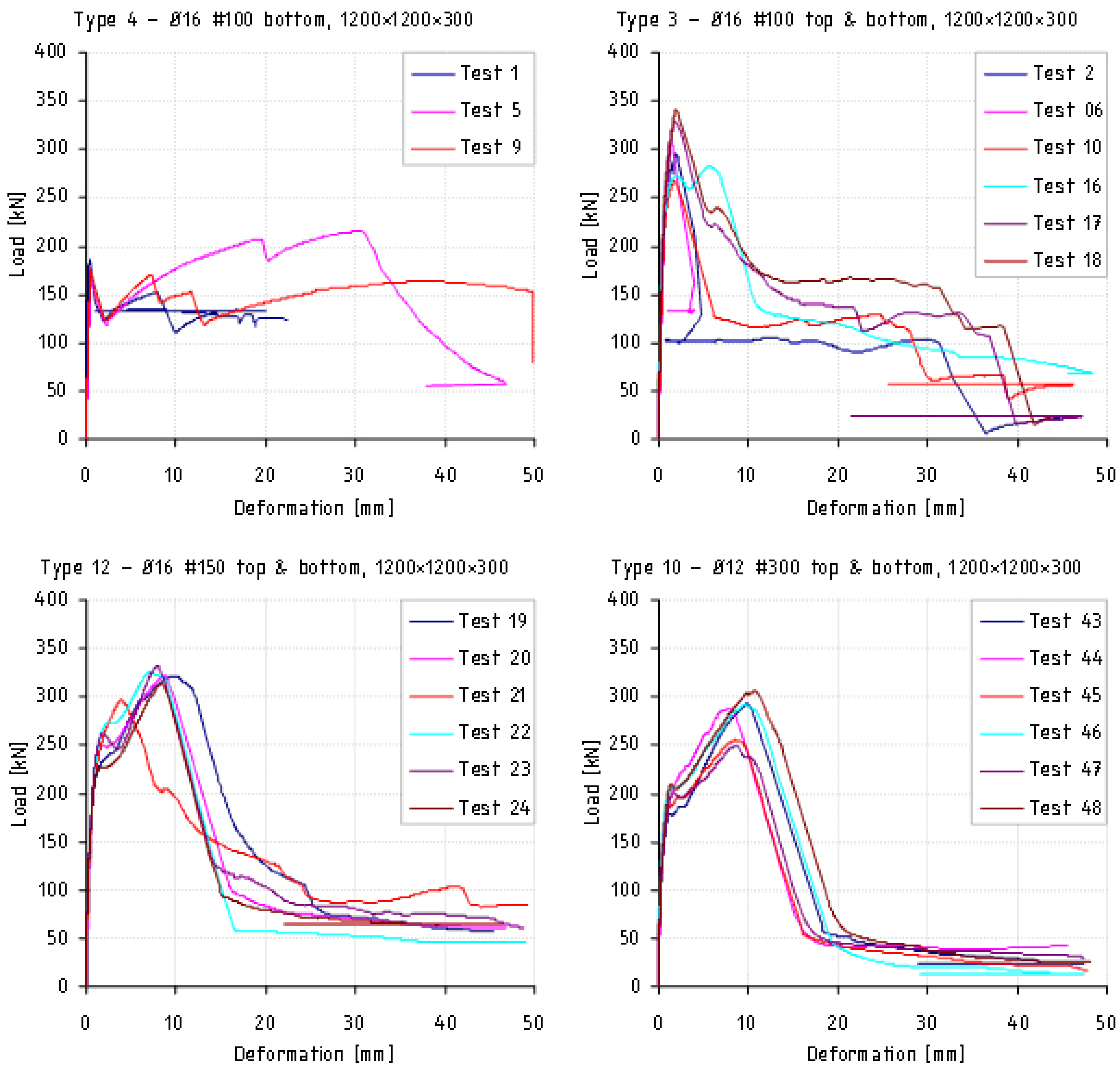

Figure 5 – Test results for slabs without cracks.

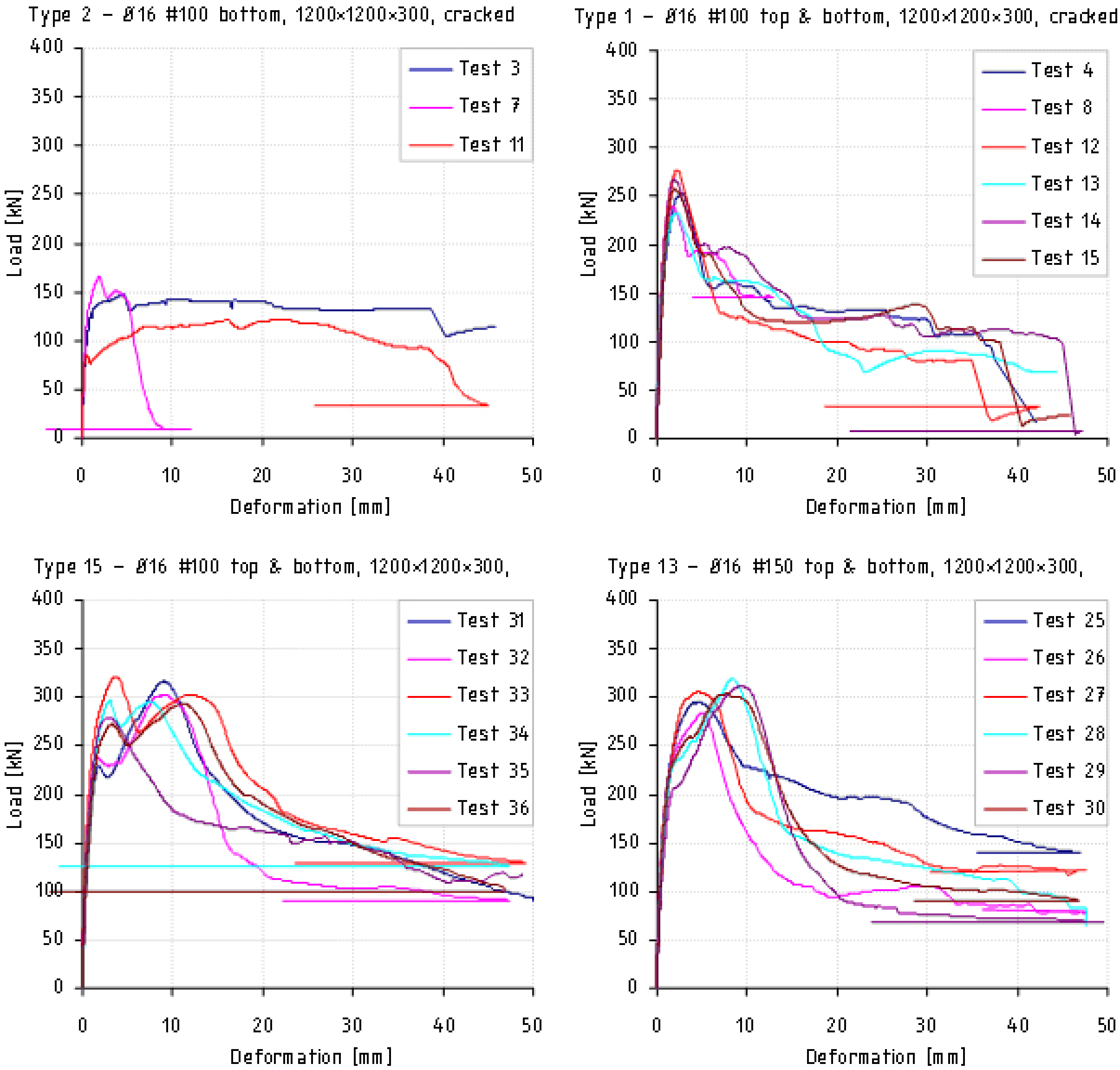

Figure 6 – Test results for slabs with cracks.

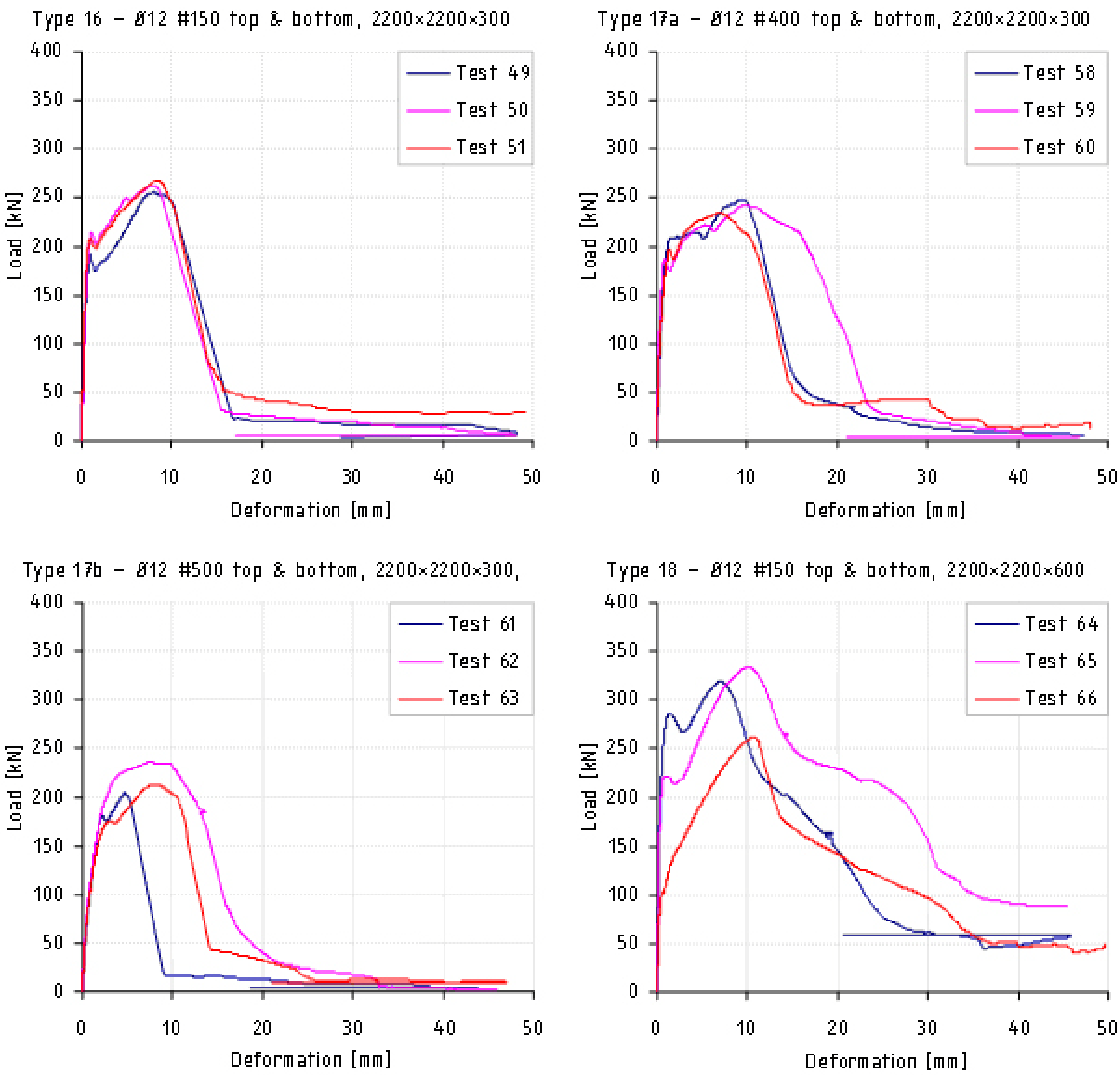

Figure 7 – Test results for larger slabs (2.2 m × 2.2 m), varying amount of top reinforcement and slab thickness.

5 ANALYSIS OF TEST RESULTS

5.1 General

The capacity for concrete cone failure of cast-in headed studs in concrete structures has been tested in order to check the influence of surface reinforcement and other boundary conditions. Ordinarily the failure load is given according to Eq. (1)

$$P_c = k f_{c,cyl}^{0.5} h_{ef}^{1.5}$$

where k is an empirical factor given in several codes. For un-cracked and cracked concrete, characteristic values for k are 12.7 $(N/mm)^{0.5}$ and 8.9 $(N/mm)^{0.5}$ respectively.

5.2 Influence of surface reinforcement

For the specimen without surface reinforcement it can be calculated that the ratio of tested to code capacity P_{test}/P_{code} is less than 1. This is due to the fact that a splitting failure occurred before a cone failure could form. If the slab is thicker (or reinforced) this failure type will be avoided.

For the slabs with surface reinforcement and without cracks the ratio of P_{test}/P_{code} is mostly higher than 1. The ratio is 0.90 for large, thin slabs 2.2 m × 2.2 m × 0.3 m where the stiffness is low causing bending deformations and cracking which reduce the capacity. The slab does not achieve a regular concrete cone failure but is also influenced by a splitting failure and the numbers have therefore been put in parentheses. If the reinforcement is cleared away in an area around the bolt of 0.4 m × 0.4 m, the ratio is further reduced to 0.84. On the other hand, when the thickness of the slab is increased from 0.3 m to 0.6 m, the P_{test}/P_{code} ratio increases from 0.91 to 1.14.

For the slabs with surface reinforcement and with cracks the ratio of P_{test}/P_{code} is between 1.23 and 1.54. The lowest value is occurring for the highest reinforcement ratio which might look confusing. The reason is probably that a high reinforcement ratio makes the slab stiff so that few additional cracks appear before maximum load. For a lower reinforcement ratio the slab will deform more and the reinforcement will have a larger dowel action. The ratio of P_{test}/P_{code} is reduced to 1.05 if the reinforcement is cleared away in an area of 0.5 m × 0.5 m around the bolt. It can also be seen that a thicker reinforcement concrete cover c = 50 mm (instead of c = 30 mm) has a positive influence and increases the ratio from 1.23 to 1.54 for otherwise similar bolts. Some of the slabs, originally without cracks, obtained considerable bending cracks during the testing. It is therefore interesting also to use the design formula for cracked slabs when these slabs are evaluated. It is found that the ratio P_{test}/P_{code} gets a value of 1.29 for large slender slabs with a minimum reinforcement of 0.15% to be compared to the value 1.40 for the smaller slabs. It can also be seen that the ratio is reduced to 1.19 if the reinforcement is cleared away in an area of 0.4 × 0.4 m around the bolt. If also the two thick large originally uncracked slabs are evaluated in this way the ratio get a value of 1.62.

The load bearing capacity for cracked slabs are somewhat smaller for high reinforcement content P_{test}/P_{code} = 1.23 (∅16 #100) than for slabs with lesser reinforcement content P_{test}/P_{code} = 1.47 (∅16 #150) and P_{test}/P_{code} = 1.39 (∅12 #300). This might be due to the fact that a much higher load is needed to obtain a crack in a more heavily reinforced slab than in a less reinforced slab. This causes higher stresses.

The test with L = 2.2 m and an area free from reinforcement around the anchor bolt showed a smaller capacity than other tests. For the un-cracked slabs we obtained P_{test}/P_{code} = 0.90 for ∅12 #300 but only P_{test}/P_{code} = 0.84 when we arranged a reinforcement free space of 400 mm × 400 mm in a slab with ∅12 #150. For a cracked slab we got P_{test}/P_{code} = 1.05 when we arranged a reinforcement free space of 500 mm × 500 mm in a slab with ∅12 #150. This indicates that it is important that the failure cone is crossed by surface reinforcement.

Earlier tests on the influence of surface reinforcement have been made by [22]. They tested expansion anchors of size M10 – M20 and found that a surface reinforcement had little influence on the load-carrying capacity for cone break-out failures. It did have the effect that the failure got more ductile. They also found that it had a positive influence on the splitting failure. Some tests have also been presented by [23] in a study of fastenings in the concrete tensile zone.

5.3 Influence of initial splitting cracks

We can obtain a failure that is initiated by a bending crack before the concrete cone failure develops. For a bending crack we need a bending moment m_{cr} [Nm] for a slab strip (with the width b = 1.0 m), the height h = 0.3 m and the crack strength f_{ct} = 3 MPa

$$m_{cr} = f_{ct}W = f_{ct}\frac{h^2}{6} = 3\cdot 10^6 \cdot \frac{0.3^2}{6} = 45000 \text{ Nm/m}$$

A line load p [N/m] that is applied transversally in the middle of a simply supported beam with the length L [m] and the width b [m] obtains a bending moment m [Nm/m]

$$m = \frac{pL}{4}$$

The cracking load p_{cr} for a simply supported slab strip with the width b = 1.2 m will then be

$$P_{cr} = pb = \frac{4m}{L}b = \frac{4\cdot 45000}{1.2}\cdot 1.2 = 180000 \text{ N}$$

If the slab thickness is doubled, the crack load will be four times as high P_{cr} = 720 kN. If the length L and the width b are doubled at the same time, this will not influence the crack load. However, the slab is not a simply supported beam but rather a circular slab with the diameter b. According to the theory of elasticity the bending moment will go towards infinity when a concentrated point load acts at its center. According to the yield line theory for concrete slabs the crack (yield) moment can approximately be written as, [24]:

$$P_{cr} = 2\pi m = 2\pi\cdot 45000 = 282743 \text{ N for } h = 0.3 \text{ m and}$$
$$P_{cr} = 2\pi m = 2\pi\cdot 180000 = 1130973 \text{ N for } h = 0.6 \text{ m}$$

From this it is clear that the thin slabs without surface reinforcement are influenced by a concrete splitting failure and thus get failure loads which are lower than the cone failure. This failure is prevented when surface reinforcement is present.

For the large un-cracked thin slab with surface reinforcement ∅12 #300 the capacity is about 90% of the code value. This is due to the fact that the restraints on these slabs are smaller than for the smaller slabs.

The reason why the load carrying capacity for cracked slabs with ∅12 #300 are somewhat higher than for corresponding un-cracked slabs (286 kN and 274 kN respectively) is probably due to natural variations in the concrete properties.

5.4 Influence of thickness

A thicker slab is stiffer and does not deform as much as thin slab and this increases the capacity of a fastener.

6 CONCLUSIONS AND OUTLOOK

The following conclusions can be drawn:
- There is a considerable increase in the load-bearing capacity if surface reinforcement is present
- The increase depends on the geometry and the amount and placement of the reinforcement

Further work is needed regarding
- Numerical modelling of the failure process for different boundary conditions with help of non-linear finite element methods
- Amount and spacing of reinforcement

Further work is also needed regarding the influence of surface reinforcement on
- Cast-in bars
- Bonded anchors

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