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ANCHOR BOLTS AND TENSION STIFFENING OF REBARS - TWO RILEM ROUND ROBIN INVESTIGATIONS ON BOND

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Abstract
Results from two RILEM Round Robin Investigations are presented. The first one deals with anchor bolts and the second one with bond and tension stiffening of rebars. Some 30 groups of researches encompassing over 70 people have performed more than 350 tests and analyses in America, Asia, Australia and Europe.
Key words: Anchor bolts, tension stiffening, crack modelling

1. Introduction

A technical committee on Fracture Mechanics of Concrete was formed in 1979 by the International Union of Testing and Research Laboratories for Materials and Structures (RILEM). It discussed and proposed methods of analysis and test methods (RILEM TC 50-FMC, 1985). To continue the task after the termination of this first committee, two new committees were set up in 1986, one for further work on test methods (TC 89-FMT) and one for applications (TC 90-FMA). The committee on applications was followed by a committee on bond in 1993 (TC 147-FMB).

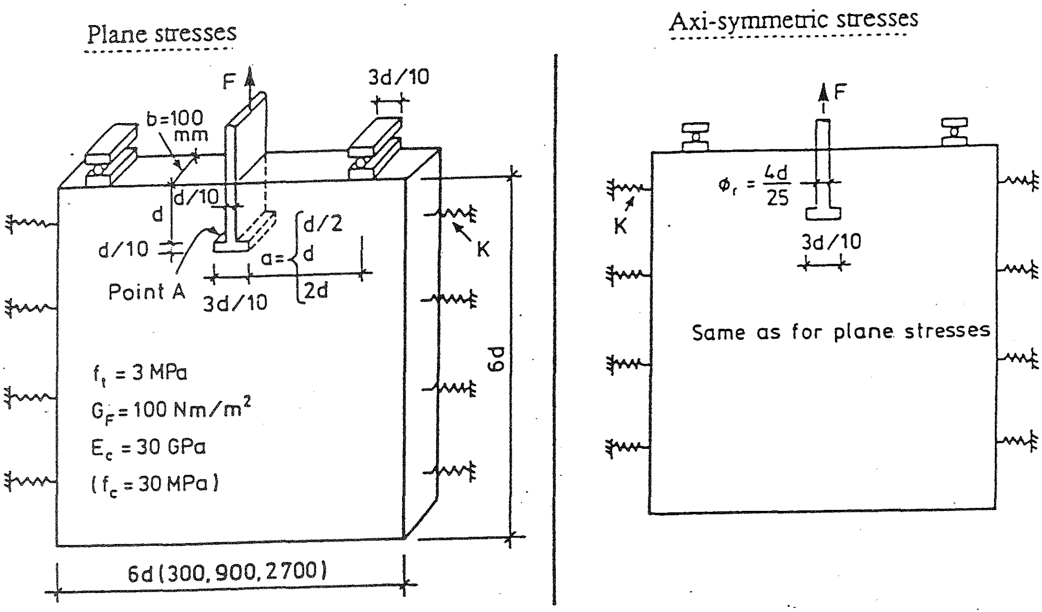
Important tasks of the committees have been Round Robin Investigations. Some highlights of the two of these investigations are presented here.

2. Anchor bolts

2.1 Background

Anchor bolts are common in concrete structures and problems regarding anchor bolts have a great practical importance. The load-carrying capacity for different types of anchors are mostly determined by testing or by simplified empirical formulae as analytical methods are either too rough or too complicated. A general review of the behaviour of anchor bolts and fastenings can be found in Eligehausen et al (1994, 1997).

The anchorage problem is a challenge from a theoretical point of view. It is intriguing due to the presence of shear as well as normal stresses. However, it should be possible to analyse it with recently developed numerical methods based on for example fracture mechanics methods.



(a) (b)

Figure 1. Invitation to Round Robin Analysis and Tests of Anchor Bolts. Analyse and/or test any or both of the problems in Fig (a) plane stresses, and Fig. (b) axi-symmetric stresses. Give the following results: (a) Description of model of analysis and/or test method; (b) Peak load; (c) Displacement of upper edge of anchor (Point A in Fig. (a)) at peak load; (d) Failure mechanism. If possible, please also give: (e) Load - displacement curve; (f) Crack pattern at peak load and at termination of analysis/test; and (g) load at onset of cracking. From RILEM TC 90-FMA (1991).

Table 1. Recommended values of parameters in Fig. 1. If you can only analyse one case, please choose the one marked with a bold **X**.

Plane stresses

K = 0				K = ∞
d =	50	150	450	150
a = d/2		x		
a = d		x		x
a = 2d	x	**X**	x	x

Axi-symmetric stresses

K = 0		K = ∞		
d =	150	50	150	450
a = d/2			x	
a = d	x		x	
a = 2d		x	**X**	x

2.2 Invitation

Based on discussions in RILEM TC 90-FMA (Fracture Mechanics of Concrete - Applications) in Sendai, Japan, 1988, and in Cardiff, Wales, 1989, an invitation was issued in 1989/90 to a Round Robin Analysis of Anchor Bolts (RILEM TC 90-FMA 1990, 1991). The test configuration in Fig. 1 for *plane stresses* was based on a proposal made by Arne Hillerborg and was similar to a test set-up that had earlier been used by Surendra P. Shah and Robert Ballarini (Ballarini et al 1986). The idea was to use a fairly simple problem which could be used to compare different numerical methods and computer codes and which should also be possible to test in most laboratories. The test configuration for *axi-symmetric stresses* in Fig. 1 is similar to what is commonly used for testing of anchor bolts.

Sixteen contributions were submitted in 1990. There was quite a scatter in the results and it was decided to give more precise rules for what to be calculated and what to be presented. It was also decided to invite to actual testing of the proposed loading cases. A second invitation was issued in 1990/91, see Fig. 1 and Table 1. (In it, the following changes were made in the axi-symmetric problem compared to the first invitation in order to make the tests more realistic: the span-depth ratio a/d was changed from 3 to 2 and the first choice of horizontal stiffness was changed from $k = 0$ to $k = \infty$).

In 1991 the Japanese Concrete Society, JCI, also initiated Round Robin Tests and Analysis of Beams and Anchor Bolts. The results have been reported by Shirai (1994). The results regarding anchor bolts are included here as well.

In order to make the results available to a wider audience it was decided to publish the contributions. This publication has been much delayed but is now finally to be published, Elfgren et al (1998).

2.3 Contributions

A list of all contributions is given in Table 2.

Table 2. Overview of Contributions

No	Contributors	Plane Stress Problem		Axi-symmetric Problem		Date
		Analyses	Tests	Analyses	Tests	
1	Alvaredo, Slowik, Wittmann	--	25	--	--	92
2	Barr, Tokatly			1	4+(15)	90
3	Bittencourt, Ingraffea, Llorca	2	--	-	--	92
4	Braestrup	2	--	6	--	91
5	Cervenka, Pukl, Eligehausen	14	--	--	--	91
6	Clement, Mazars	1+2	--	1	--	91
7	Eligehausen, Bouska, Cervenka, Pukl	--	--	--	9+(26)	92
8	Fathy, Planas, Elices, Guinea	--	6+(18)	--	--	92
9	Hassanzadeh	1	--	--	--	90
10	Karihaloo	--	12	--	8	92
11	Leonhard, Rots, de Borst, Feenstra	7	--	4	--	91
12	Manfroni, DiTommaso	3	--	--	--	92,93
13	Merabet, Fleury, Reynouard	3	--	--	--	92
14	Ohlsson, Elfgren, Olofsson	4+(20)	8+(12)	--	5	92,93
15	Ozbolt	7	--	6	--	92
16	Palm, Gylltoft	2	--	1	--	92
17	Pankaj, Bicanic	1	--	--	--	90
18	Pukl, Margoldava, Bouska	--	--	4	--	92
19	Rossi, Wu	3	--	--	--	91
20	Shirai et al (See below)	9+(10)	13+(16)	--	--	93
21	Stork, Reinhardt	1	--	1	--	90
22	Uchida, Rokugo, Koyanagi	30	--	--	--	92
23	Valente, G, Di Tommaso	6	--	6	--	92
24	Valente, S, Carpinteri	--	--	6	--	92
25	Vervuurt, Schlangen, van Mier	10+(3)	18	--	--	93
26	Wang, Navi, Huet	7	--	-	--	92
27	Yankelevsky, Leibowitz	6	--	6	--	92
Total for 27 contributions		121+(33)	82+(46)	42	22+(45)	

Notes:

- Numbers in parentheses refer to analyses and tests with geometries which deviate from the invitation in one respect or the other.
- Contributions with date 90 were submitted as an answer to the first invitation and were discussed in Torino in October 1990. Contributions with date 91 were submitted as an answer to the revised invitation and were discussed in Delft in June 1991. Contributions with later dates have been revised or are new additions.
- The following persons contributed to the Japanese Benchmark on Anchor Bolts, Shirai et al [20]: **Tests**:T2-1: Y. Uchida, K. Rokugo and W. Koyanagi; Gifu University; - T2-2: K. Nakagawa; Tobishima Corporation, and K. Maruyama; Technological University of Nagaoka; - T2-3: N. Nakajima, Y. Shinozaki and H. Mikami; Mitsui Construction Co., Ltd.; - T2-4: Y. Maki, K. Nishimura, T. Horiuchi; Hosei University, and T. Nukariya; Chichibu Onoda Cement Co., Ltd.; - T2-5: K. Yamada, K. Watanabe, Y. Asai; Aichi Institute of Technology, T. Yamamoto, M. Ishikawa; Tokyu Construction Co., Ltd., and U. Kanetoh; Total Information Service. **Analyses**: A2-1: J. Ishida; Konoike Construction Co., Ltd.; - A2-2: K. Itoh; CTI Engineering Company Ltd., and T. Taniguchi, S. Hirose and S. Matsunaga; Okayama University.; - A2-3: M. Ueda, A. Kambayashi; Takenaka Corporation, N. Takeuchi, M. Takano; Meisei University, H. Kitoh; Osaka City University, and H. Higuchi; Abe Kogyo-sho. ;

- A2-4: Y. Uchida, K. Rokugo and W. Koyanagi; Gifu University; - A2-5: Y. Ohtani; Kobe University, and Chong Kyong Ok; Oriental Construction Co., Ltd.; -. A2-6: Ali Hassan Chahrour and M. Ohtsu; Kumamoto University; - A2-7: N. Nakajima; Mitsui Construction Co., Ltd.; - A2-8: T. Shiraishi, John Bolander Jr. and H. Hikosaka; Kyushu University; - A2-9: T. Miyashita and Y. Hayami; Kajima Corporation; - A2-10: K. Yamada, K. Watanabe, Y. Asai; Aichi Institute of Technology.T. Yamamoto, M. Ishikawa; Tokyu Construction Co., Ltd., and U. Kanetoh; Total Information Service.

2.4 Summary of results

Some of the results are illustrated in histograms given in Fig. 2 showing the distribution of maximum loads for the main cases of the *Plane Stress Problem* and the *Axisymmetric Stress Problem.* It can be seen that the mean value of the loads is 345 kN/m in the tests and 427 kN/m in the analysis for the *Plane Stress Problem* and 190 kN in the tests and 227 kN in the analysis of the *Axi-symmetric Stress Problem.* The standard deviation is 69 and 97 kN/m and 53 and 69 kN respectively. The deviation is rather big both in the tests and in the analyses

Some *crack patterns* are illustrated in Fig. 3. Cracks usually start to grow on both sides of the anchor bolt. If horizontal restrictions are present, such as lateral confinement and/or friction in the test set-up, they tend to cause the cracks to be symmetric in the beginning. When the cracks develop further they turn asymmetric. This is often the case from the beginning if no restraint is present. The failure is mostly caused by a horizontal crack or by a crack growing in a direction opposite to the support. Typical *load displacement-curves* are shown in Fig. 3.(d), where the influence of the embedment depth *d* is shown. The curves are smooth and do not show any snap back behaviour.

The differences in the *test results* can be due to varying loading and support systems and to varying concrete properties. The discrepancies in the *analyses* can be due to differences in:

- experiences of non-linear modelling of the different analysers,
- constitutive models and computer codes with different treatment of softening (local, non-local or discrete),
- assumed material parameters (e.g. the shape of the softening diagram was not prescribed),
- mesh finess and mesh orientation,
- treatment of local mode II (shear) effects at integration point level,
- modelling of boundary conditions (as e. g. the load introduction via the steel bolt; the effect of the mesh lay-out in this region is a delicate matter),
- the over-all stiffness of the model tends to be too high due to a combination of the factors above.

Further problems in this specific case have been pointed out by Tano (1997): (a) There is nearly a hydrostatic stress state in the region through which the crack will propagate. This will lead to bad initial crack directions in some elements, particularly if a local failure criteria is used. (b) There exists two different failure modes, one symmetric and one

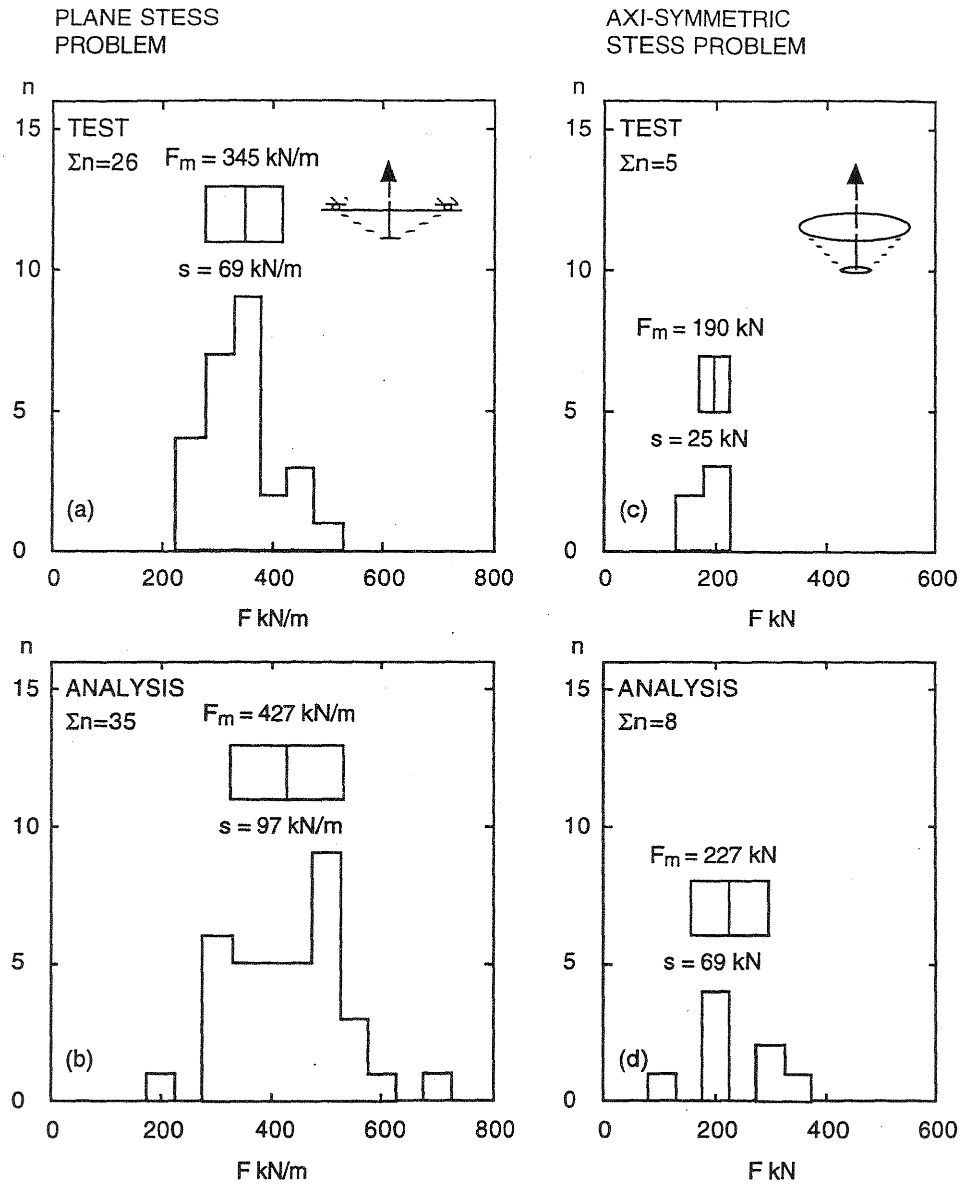


Fig. 2. Histograms showing numbers of contributions with maximum loads in different load intervals. (a)-(b) Plane Stress Problem. No horizontal confinement $k = 0$, (c)-(d) Axi-symmetric Stress Problem. Horizontal confinement $k \neq 0$. Embedment depth $d = 150$ mm, span/depth ratio $a/d = 2$. The results are normalised to a concrete compressive strength $f_c = 30$ MPa. The mean value F_m of the loads is 345 kN/m in the tests and 427 kN/m in the analysis for the *Plane Stress Problem* and 190 kN in the tests and 227 kN in the analysis of the *Axi-symmetric Stress Problem*. The standard deviation s is 69 and 97 kN/m and 53 and 69 kN respectively.

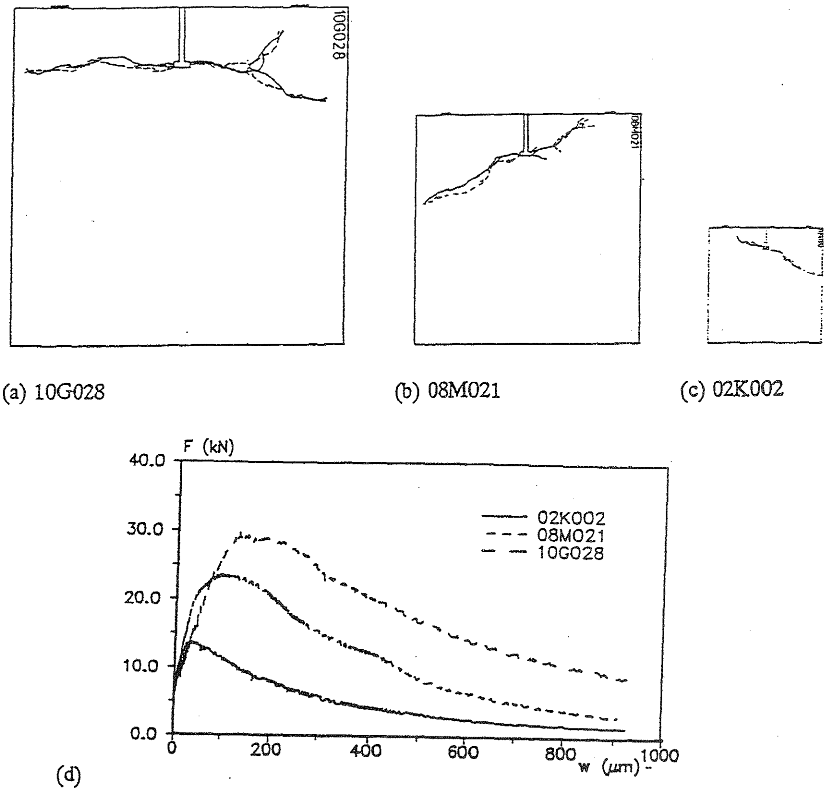


Fig. 3 Crack patterns as observed by Verwuurt et al [25] (1994) for three specimen with span/depth ratio a/d = 2 and with three different sizes: (a) d = 150 mm, (b) d = 100 mm, and (c) d = 50 mm. The corresponding load-displacement responses are shown in (d). The width of the specimens were 100 mm.

nonsymmetric and it is not obvious for all geometries which one will dominate. (c) Crack branching is likely to occur and the model must be able to cope with that.

The *size dependence* of the problem is illustrated in Fig. 4. Here the failure stresses for specimens with embedment depths d = 150 and 450 mm are compared to the failure stress for specimens with embedment depth d = 50 mm. There is quite a scatter in the test results in the upper diagram if all curves are considered. However, only the two upper curves are really representative. The following two curves refer to other materials (mortar and lightweight aggregate) and the last one has a horisontal restraint.

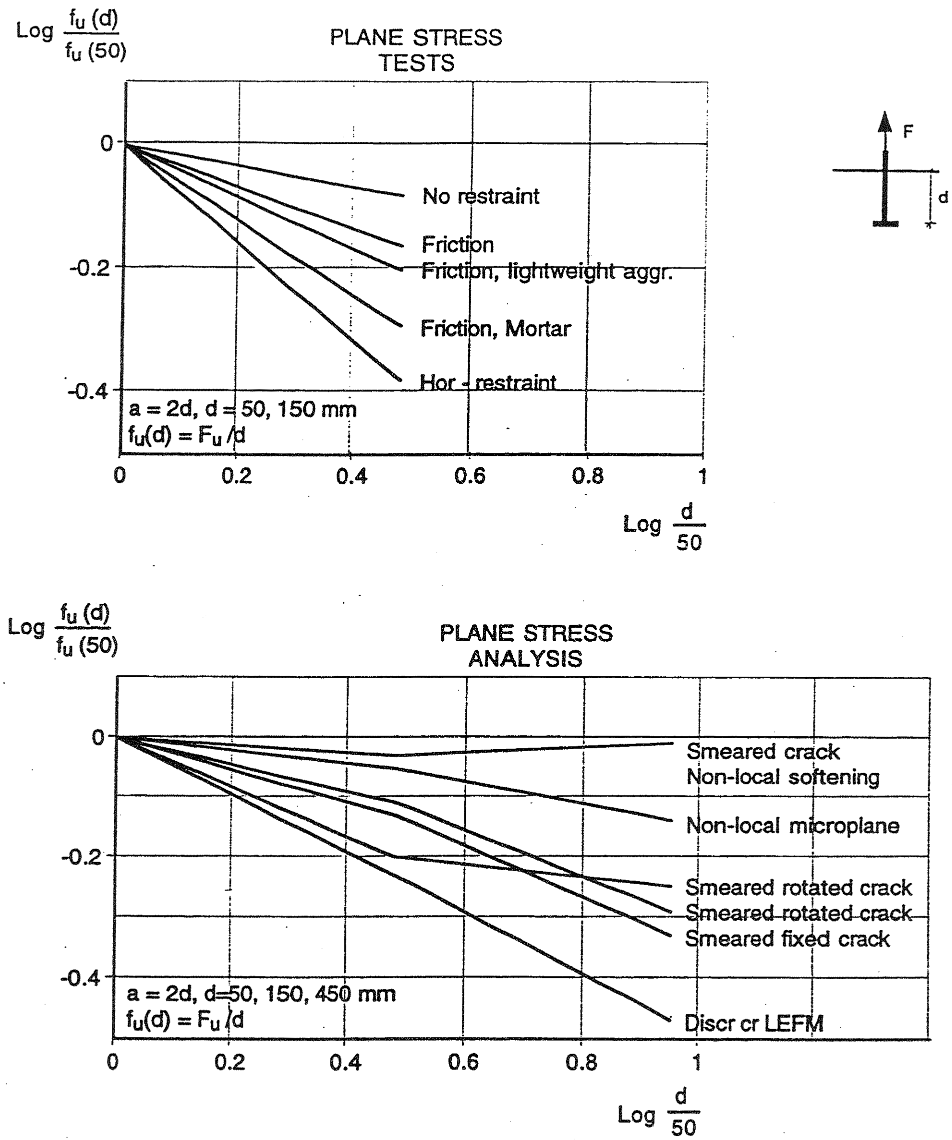


Fig. 4. Size dependence of nominal failure stresses as function of relative embedment depths d.

In the lower diagram of Fig. 4 the analytical results can be compared. Some of the methods based on smeared cracks and on non-local microplanes predicted the size dependence quite well.

Fewer groups took part in the *Axi-symmetric Stress Problem.* There was a somewhat smaller deviation especially in the test results probably due to a more well-defined, traditional test set-up.

3. Tension stiffening of rebars

3.1 Background

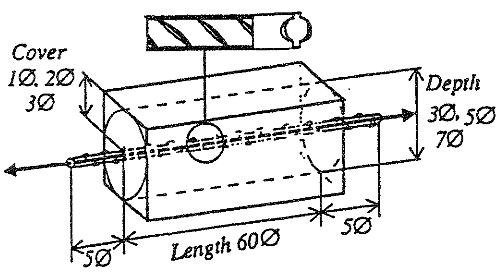
The action of a tensioned rebar embedded in concrete is stiffened by the surrounding concrete due to the bond between the materials. This stiffening is fundamental for the correct modelling of reinforced concrete structures. Based on discussions in RILEM TC 147-FMB an invitation was issued in 1996 for Round Robin Analysis and Tests on the behaviour of a deformed bar cast in a concrete prism according to Fig. 5. An axial, tensile force was to be applied to the bar until collapse of the specimen. The following results should be given:

- Description of model of analysis and/or test method
- Material properties at the time of testing/analysis
- Load-displacement curve (peak load, relative displacement of the bar, post-peak response)
- Crack patterns and crack widths and corresponding mean strain of reinforcement bar
- Failure mechanisms

3.2 Contributions

The following contributions have been submitted so far (January 1998):

- S. Al-Fayadh, J. Magnusson, B. Engström, Chalmers University of Technology, Sweden [1] - Tests and analyses with 12 and 16 mm bars.
- M. Arduini, Bologna University, and S. Russo, Venice Institute University, Italy [2]- Tests with 8 and 16 mm bars.
- R. Eligehausen, A. Bigaj, U. Mayer and F. Sanchez, University of Stuttgart, Germany [3] - Tests and analyses with 16 mm bars.
- H. Hikosaka, Y. Liu and S. Saito, Kyushu University, Japan [4]- Analytical results for 3 geometries.



Concrete	NSC			HSC
Cover	1Ø	2Ø	3Ø	2Ø
Ø8		x		
Ø16	x	x	x	x
Ø32		x		

Fig. 5. The features of the specimens used in the Round Robin Analysis and Tests on Bond and Tension Stiffening. The table gives the recommended variation of parameters. If only one case can be performed, choose the circled one. From RILEM TC147-FMB (1996).

• K. Noghabai, L. Elfgren, Luleå University of Technology, Sweden [5] - Tests and analyses of 16 and 32 mm bars.
• B. Tork, J. Gálvez, J. Planas and M.Elices, University Polytechnic of Madrid, Spain [6] - Tests with 8 and 16 mm bars.

3.3 Some results

Some preliminary results are given in Table 3 and Fig. 6. Although there seem to be some differences between the tested cracking loads, the general behaviour is similar. A thicker cover gives fewer cracks. There does not seem to be a big difference between normal and high strength concrete. Analytical models based on slip as well as on FEM with inner softening bands may be able to predict the behaviour in a correct way.

4. Conclusions

4.1 Anchor bolts

In *testing* the importance of the boundary conditions was made clear. A slight change in the stiffness of the supports or of the depth of the specimen at once changes the ultimate load capacity.

In *analysis* it was clear that both discrete and smeared methods can give a variety of results depending on the analyser and what choices he/she makes regarding materials parameters (such as softening), mesh size and orientation, and crack propagation.

The *educational importance* of the benchmark has been quite important and can give guidance of how to go about to get correct results.

The *main conclusion* is that it is possible to reasonably correctly analyse both plane and axi-symmetric anchor bolt problems. However, there are many pitfalls on the road and much knowledge is required from the analyser.

Table 3. Summary of contributions and some results for a specimen with a rebar diamameter of 16 mm and a concrete cover of 32 mm

Investigator	Number of tests + analyses	NSC Crack Load Pcr kN	Compr strength fcc MPa	Crack space srm mm	HSC Crack Load Pcr kN	Compr strength fcc MPa	Crack space srm mm
1. Sweden	2x5 + 2x5	13	35	160	17	150	120
2. Italy	2.5x4 + 0	13	35	90	7	90	110
3. Germany	2x4 + 2x4	-	45	105	-	135	73
4. Japan	0 + 3	12	30	60	-	-	-
5. Sweden	8 + 2	23	60	80	23	80	120
6. Spain	3x3 + 0	27	35	120	-	-	-

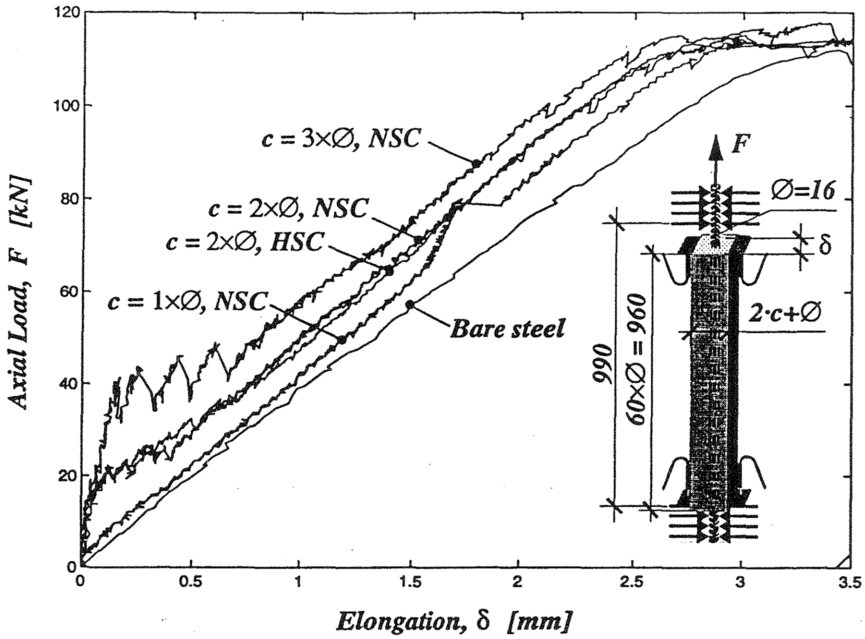


Fig 6. Typical load-deformation graph for increased tension stiffening effect with increased concrete cover. The influence of the concrete strength is limited. From Elfgren and Noghabai (1998). See also Noghabai (1998).

4.2 Tension stiffening

All results have not been reported and analysed yet. However, similar trends were seen in the different tests and most of the applied theoretical models give reasonable results.

4.3 Acknowledgements

The work performed by all the persons who have contributed to the investigations are greatly appreciated. Rolf Eligehausen and Jan Rots are acknowledged for being members of the editorial group for the Anchor Bolts Report, Elfgren et al (1998).

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