International Journal of Current Engineering and Technology
E-ISSN 2277 – 4106, P-ISSN 2347 - 5161

Available at http://inpressco.com/category/ijcet

Review Article

A Critical Review of Researches on Anchorages with End Hooks and Bends in Reinforced Concrete Structures

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Accepted 25 August 2014, Available online 28 Aug2014, **Vol.4, No.4 (Aug 2014)**

Abstract

This paper reviews the most experimental works on anchorages by hooked and bent bars in reinforced structure. Some of these works resulted in forming expressions to design bond strength by codes of practice like ACI. None of these works represent the real structure conditions as they were carried out on pull-out specimens except those unpublished tests by Gulparvar which applies directly to anchorages at simple supported beams. There is Schiessl's theoretical approach to treat and calculate anchorage resistance based on the of the compatibility of deformations between straight lead lengths and terminal bends and hooks.This review has been done as a part of a research study and used as a base to undertake a substantial test program on end anchorage at simply supported beams at London South Bank University between 2003-2005 by the author of this paper to obtain PhD in structural engineering. The present situation seems to be that there are 5 results by Gulparvar available from tests to failure of 90° bends at supports and none for 180° bends. There are probably isolated results from tests in which unintended anchorage failures have occurred, but the total of information is clearly very small.

Keywords: *Bearing stress, 180^o hooks or 90^o bends, tail length, concrete strength, concrete cover, transverse reinforcement, transverse pressure*

1. Introduction

If the end anchorage of a bar is assisted by a bend or hook, the action of the bend produces a concentrated compression on the concrete which produces transverse tension. Failure is often by splitting which is generally attributed solely to the bearing effect. A pull-out failure with the bar slipping around the bend is also possible.In addition to the factors mentioned previously in part one in relation to bond others which should be considered in relation to bent anchorages are:

- The considerable movement of the bar corresponding to the compression within the bend, which may make it impossible for the resistance of the bend to be fully mobilized while.
- The bond in the lead length is still intact.
- The uncertainty about the distributions of the bond and bearing stresses around a bend.
- The possibility of the splitting effects from bond and bearing being combined and so reducing the resistance to splitting.

2.Pull-out experimental works

2.1 Mylrea(Mylrea T.D.1928)

The first tests of end hooks that seem to be of any real interest appear to have been those by Mylrea. His type of test specimen is illustrated by Fig.(1). It is to some extent a model of a beam end, but the reaction R on the left produces no transverse pressure in the area of the lead length and hook. The bars were plain round with a diameter of 12.7 mm. The internal radius of bend varied from zero, i.e. a bar bent flat against itself, up to 11φ. In ten specimens there was no other reinforcement. In twelve the hook was surrounded by a helix with 5,7 or 10 turns of 6.3 mm wire. The concrete cylinder strength varied from 13.6 to 24.6 N/mm^2.

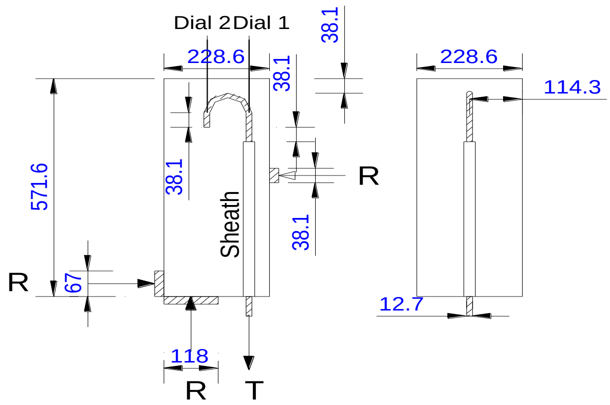


Fig.1 Mylrea's test arrangements

In all the tests the steel stresses exceeded the elastic limit. Some were terminated at large movements and in some the bars fractured where the wire used for slip movement was

*Correspoding author **Rizgar Amin Agha (BSc, MSc, PhD)** is working as Senior Lecturer fixed, but the majority failed by splitting. Where the bends had normal internal radii ($\geq 2.5\varphi$), the movements at both the dial gauges shown in Fig.(1) were around the bend, i.e. upward at gauge 2 and downward at gauge 1. However for $r=\varphi$, and in late stages for $r=\varphi$, at gauge 2 the bar moved downward and the end bearing of the bar bent the whole anchorage.

In calculating nominal bearing stresses $\sigma_{b,nom}=F/A_b$, Mylrea included the end face of the bar in the bearing area for $r=\varphi$and $r=\varphi$, making $A_b=\pi\varphi^2/4+2r\varphi$. For larger radii of bend $A_b=2r\varphi$.Thus, for a given value of F , his nominal bearing stresses were much lower than those calculated by current methods ($\sigma_b=F/r\varphi$).Mylrea's calculations assumed a high bar stiffness and flexural strength, while present methods ignore the bending stiffness and strength.The present approach is reasonable for large radii of bend, but bar flexure does appear to play a role for small radii- as r approaches zero, σ_b does not tend to infinity. In Mylrea's tests the resistances of anchorages with $r=\varphi$ and $r=\varphi$ were practically the same as those for $r=2.5\varphi$

2.2 Muller(Muller.H.H.1968)

Muller made 500 tests of which 300 were on ribbed bars with terminal hooks or bends. The bar sizes were 8,12 and 18 mm and the tests specimens were 200 mm cubes for the two smaller bar sizes and 300 mm cubes for the 18 mm bars.The bars had straight bonded lengths $=2\varphi$ before and after the bends.

2.3 Hribar and Vasko(Hribar J.A et al. 1975)

In the tests by Hribar and Vasko hollow-ram jacks reacting against the top surfaces of concrete blocks were used to apply tension to anchorages embedded in the blocks.The bars were debonded from the surface down to the levels of the beginnings of the anchorages, which were at the starts of the curves of 90° bends and 180° hooks.The bars had stress-strain curves which were linear up to about 400 N/mm^2 and had a well-defined yield at about 422 N/mm^2 .The ultimate stresses were about 650,680 and 730 N/mm^2 for bar sizes of 12.7, 22.2 and 34.9 N/mm^2 . Their deformations complied with the provisions of ASTM A305-56T which would appear to satisfy the requirements of EC2.

The tests were continued until failure which was generally by the fracture of the bars.The corresponding bar stresses are not given in most cases- the ultimate stresses quoted above are average values from the tests for which ultimate stresses are given. Stresses corresponding to givendisplacements and displacements corresponding to given stresses are detailed in (Hribar J.A *et al*. 1975). The displacements or slips given were obtained by subtracting the elastic elongation of the bar between the measuring point at the start of the anchorage from the total displacement between the measuring point and the concrete surface which would seem to involve significant errors once the bar stresses exceeded the elastic limit.

The two main series of tests (series II and III) were made on bars cast in two large concrete blocks approximately 1.5 m deep and 4.6 m square(series II) or 3.04x3.66 m (series III). In these tests the depths to the starts of the anchorages were generally 254, 457 and 616 mm for 12.7,22.2 and 34.9 mm bars. These depths were such that the calculated pressure in the concrete was about 0.5 N/mm^2. In series I some tests were made in a large plain concrete block but others were in small blocks (of unknown dimensions) with heavy, but undefined reinforcement. Depths to the starts of the bonded lengths varied but are not defined. Because of uncertainties here, these tests are generally not considered in the following.

The main conclusions drawn by Hribar and Vasko are

1. A well-defined relationship exists between average bond stress and bonded length for each bar size and is relatively insensitive to the type of anchorage (straight or bent). Although this is not stated in (Hribar J.A *et al*. 1975) this conclusion relates to average stress at a given slip(principally 0.25 mm). In the few cases where pull-out failures occurred, the ultimate bond stresses were practically independent of bar size. The relationships drawn in (Hribar J.A *et al*. 1975)show the bond stress increasing for increasing bar size. This is misleading, as a more rational basis for comparison would be the bond stresses for different bar sizes in tests with equal l_b/φ (not equall_b). From the paper's Fig.2, which shows bond stresses at 0.25 mm slip,for bar sizes of 12.7, 22.2 and 34.9 mm , the bond stresses at $l_b/\varphi=10$ are 9.4,9.0 and 8.8 N/mm^2 and those at $l_b/\varphi=20$ are 6.2,4.7 and 5.5 N/mm^2 .Thus for similar values of l_b/φ the bond stress for a given slip decreases relatively slightly, with increasing bar size.

2. Hook geometry and embedment length were the prime factors affecting the pull-out characteristic of a bar.Prior to yielding the efficiency of an anchor decreased with increase in hook angle, but beyond yield of the steel a 180° hook performed better than a 90° hook. An increase in radius of hook increased the efficiency of an anchor more than an extension beyond the hook.

3. Point (1) above suggests that anchors, curved or straight, can be designed on the basis of bond alone.

4. The results are based on tests in which splitting was prevented by either mass concrete or (in series I only) auxiliary reinforcement and would be applicable to such situations. The value of an anchor should be based on the ultimate concrete strength and the restraint against splitting. A distinct linear relationship exists between stress and loaded end displacement at low bar stresses (100 N/mm^2 for 34.9 mm bars to 240 N/mm^2 for 12.7 mm bars). Deviation from a linear relationship indicated incipient crushing of the concrete or incipient bond failure, which could significantly alter the effectiveness of anchors at high working stresses under fatigue conditions.

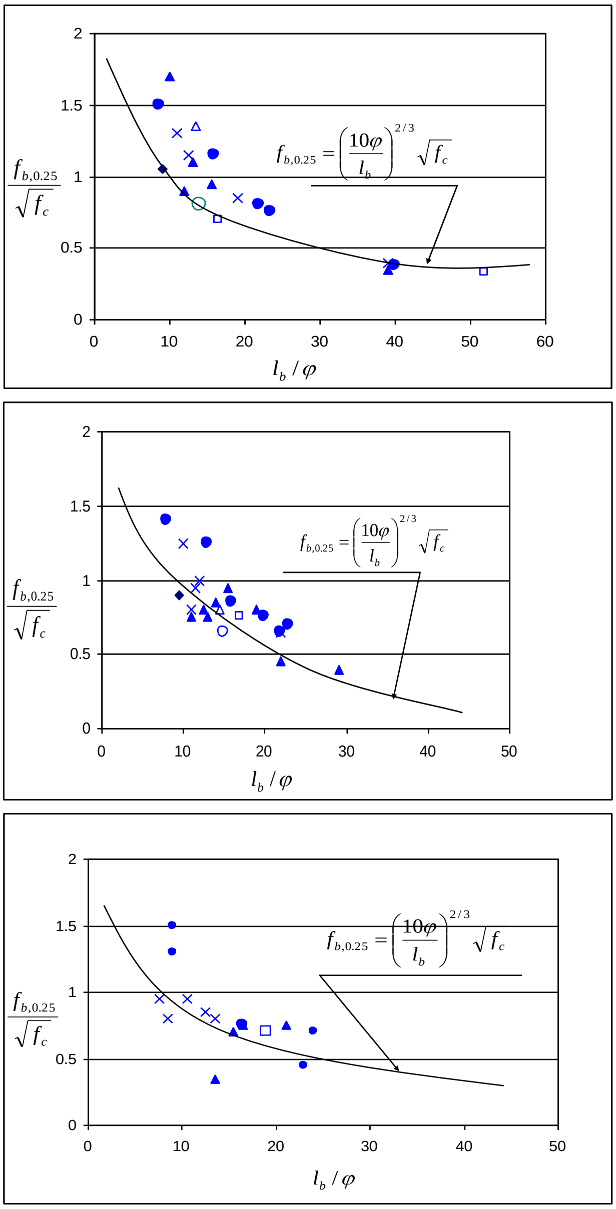
Points (3) and (4) seem confused. In view of (4) it is clear that anchorages should not be designed on the risk of their causing splitting. A rather more rational interpretation of the test results would seem to be as follows:

1. For the serviceability limit state, the steel stress corresponding to 0.25*mm* slip at the loaded end of an anchorage can be calculated assuming an average bond stress along it of

$$f_{b,0.25} = k\left(\frac{\varphi}{l_b}\right)^{2/3}\sqrt{f_c}\ \text{in SI units.} \qquad (1)$$

An average value of $k \approx (10)^{2/3}$

This relationship is compared with the test results from series II and III in Fig.(2). It should be noted that it is derived from tests with effectively infinite cover. It should however apply more generally providing the cover does not split at the load in question, which is probable at service loads,if uls requirements are met.Also it applies to first loading and the slip could be expected to increase with repeated loading.



KEY

Straight anchorages ●

90^0 bends with standard radii and t=4φ ◆

t=8φ △

t=12φ □

other 90^0 bends ×

180^0 bends with standard radii and t=4φ or 5 φ ○

other 180^0 bends ▲

Fig. 2 Average bond stresses at 0.25mm loaded-end sliptest series 1,2 and 3 by Hribar and Vasko

2. The test data are insufficient for any definitive conclusions to be drawn for uls design. No splitting failures occurred in spite of the bearing stresses in bends being up to $7f_c$, $6f_c$ and $5f_c$ for the standard bend radii of 2.5φ ($\varphi = 12.7mm$), 3.0φ ($\varphi = 22.2mm$) and 4.0φ ($\varphi = 34.9mm$). All of these are in excess of maximum characteristic resistances obtained from EC2 and BS8110 by setting $a/\varphi = \infty$ in their equations.

There were a few pull-out failures (not accompanied by any visible cracking of the blocks and the results for these are shown in Fig.(3), where the average bond stress at failure is plotted against l_b/φ .The results include the bond failures of the series I tests of bars cast in a large block. For $l_b/\varphi \geq 10$, there appears to be a fairly clear relationship between $f_{bu}/\sqrt{f_c}$ and l_b/φ, valid for both straight and bent anchorages. This could be expressed in a form similar to that used above for $f_{b,0.25}$, i.e.

$$f_{bu} = 2.25\left(\frac{10\varphi}{l_b}\right)^{2/3}\sqrt{f_c} \qquad (2)$$

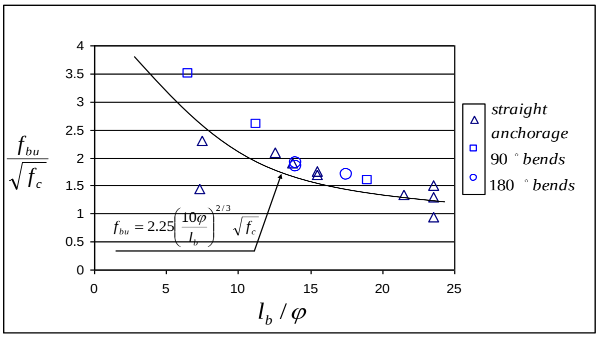


Fig. 3 Ultimate bond stresses, tests by Hribar and Vasko

In relation to both of the above conclusions, the effect of concrete strength on bond resistance has been assumed to be one of proportionality between f_b and $\sqrt{f_c}$ .The test results do not contradict this, but the variation of f_c was very limited - f_c =29.0 , 32.7 and 25.5 N/mm^2 respectively for series I, II and III. The possibility of direct comparison between Hribar and Vasko and Muller is very limited, but some observations can be made.

1. Table (1) gives Hribar and Vasko's results for the steel stresses developed at loaded end slips of 0.2 *mm* in tests of 180^o hooks with varying tail lengths, f_c was 32.75N/m^2. While Muller's conclusion that tail lengths have no

Table 1 Effect of tail length on bar stresses at slip of $0.25mm$ (Hribar and Vasko)

Specimens	2-14	2-15	2-16	2-20	2-21	2-22	2-26	2-27	2-28
$\varphi(mm)$	12.7	12.7	12.7	22.2	22.2	22.2	34.9	34.9	34.9
r/φ	2.5	2.5	2.5	3.0	3.0	3.0	4.0	4.0	4.0
l_t/φ	0	2	5	0	2	4	0	2	4
$f_{s,0.25}(N/mm^2)$	219	212	218	157	196	218	103	214	260

Table 2 Effect of radius of bend on bar stresses at slip of $0.25mm$ for bars with $l_t = 0$ (Hribar and Vasko)

Specimens	2-14	2-39	2-40	2-20	2-34	2-35	2-36	2-26	2-43
$\varphi(mm)$	12.7	12.7	12.7	22.2	22.2	22.2	34.9	34.9	34.9
r/φ	0	0	0	0	0	0	0	0	0
l_t/φ	2.5	3.5	4.5	3.0	3.5	4.0	4.5	4.0	6.0
$f_{s,0.25}(N/mm^2)$	219	276	307	157	202	243	303	103	312

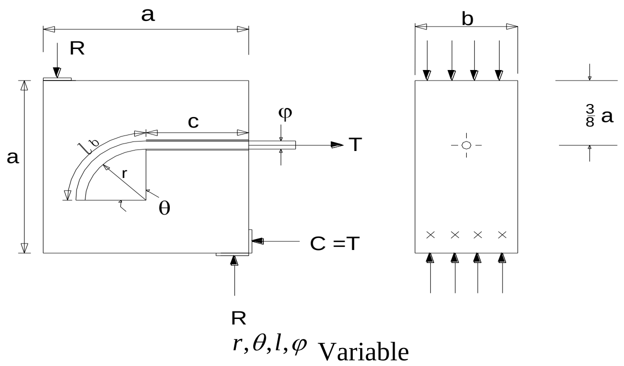
influence on bar stresses at slips up to 0.2 *mm* is supported by the results for the smallest bar size, it is clearly contradicted by those for the larger bar sizes both of which had were greater than the 18*mm* maximum size tested by Muller.

2. Muller indicates the effect of the radius of bend by a diagram valid for slips up to $0.2mm$ which shows that in relation to the bar stress for $r = 2.5\varphi$, there is a 20% reduction when $r = 1.25\varphi$ and a 25% increase when $r = 5.0\varphi$ .Table (2) gives Hribar and Vasko's results for 180° hooks with varying radii of bend (all for $f_c = 32.75$ N/mm^2). It is true that the tests by Hribar and Vasko were made on bars with $l_t/\varphi = 0$, while in Muller's $l_t = 5\varphi$, but according to Muller l_t/φ has no influence on $f_{s,0.2}$ .The results above show the bars stresses to have increased by 40% for an 80% increase of r/φ when $\varphi = 12.7mm$, by 93% for a 100% increase of r/φ when $\varphi = 22.2mm$ and by 203% for a 50% of r/φ when $\varphi = 34.9mm$. All of these increments are much greater than Muller's 25% for a 100% increase of r/φ .

2.4 Minor and Jirsa(Minor J. et al. 1975)

Minor made 38 pairs of pull-out tests to examine the influences of various parameters on the capacities of anchorages by bends.The bars were embedded in plain concrete blocks as shown in Fig.(4), with their lead lengths "c" debonded. The primary variables were the total bonded length ($l_b = 4.8\varphi$ to 9.7φ) , the angle of bend (0 to 180°), the internal radius of bend ($r = 1.6\varphi$ to 5.7φ) and the bar diameter (16,22 and 28 mm).The tail length varied as a function of l_b and r . The concrete strength (f_c) was generally in the range from 25 to 40 N/mm^2 . Measurements of slip were made at both ends of the bond lengths and at the some intermediate points.

The test arrangement seems far from ideal as the node formed by the vectors of the applied forces lies outside the bar. According to (Minor J. *et al*. 1975).



Bar size (mm)	Dimensions(mm)		
	a	b	c
15.9	305	203	152
22.3	406	203	203
28.6	406	305	190

Fig. 4 Minor and Jirsas' test arrangements

In nearly all the tests the bond lengths were short enough to produce bond failure rather than bar or concrete fracture", but 19 tests were terminated at high stress or large slip and for these the maximum loads are unknown. A further 32 failed by fracture of the concrete.This failure mode is not described but can reasonably be assumed to be splitting resulting from bond and bearing stresses.The remaining 25 tests ended in pull-out failures. In many of the tests the maximum stresses in the bars exceeded their nominal yield stress of 414 N/mm^2 .The maximum stress reported is 531 N/mm^2 .Minor and Jirsa concluded that:

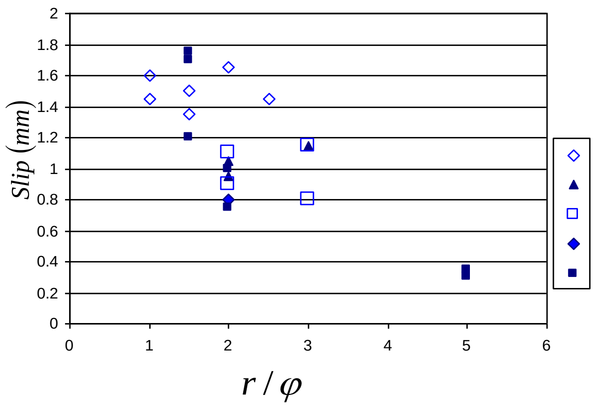
1. For equal ratios l_b/φ , the larger the angle of the bend , the greater is the slip at a given bar stress and the smaller the ratio (r/φ) , the greater is the slip at a given bar stress. Fig.(5) shows that this is broadly true for a bar stress of 414 N/mm^2 .
2. In an anchorage consisting of both bent and straight (tail) sections most of the slip is developed in the curved section.
3. There is little difference in strength between straight and bent bar anchorages except for a very short bond

lengths which would be impractical for construction applications.

The combination of conclusions 1 and 3 suggests that 90^{o}bends are preferable to180^{o} hooks and that the radius of bend should be as large as is practical to minimise slip.

Conclusion 3 suggests that bond rather than bearing was generally critical in these tests, and Fig.(6) shows $f_{bu}/\sqrt{f_c}$ plotted against l_b/φ for all the tests that ended in failures. Excepting the two tests of very short straight anchorages, the scatter of the results is not too large and there is a trend for $f_{bu}/\sqrt{f_c}$ to increase as l_b/φ decreases, although a simple linear relationship would apply to these tests in their limited range of l_b/φ, Fig.(6) plots the relationship used above for Hribar and Vasko's results and can be seen to fit Minor and Jirsa's data quite satisfactorily.

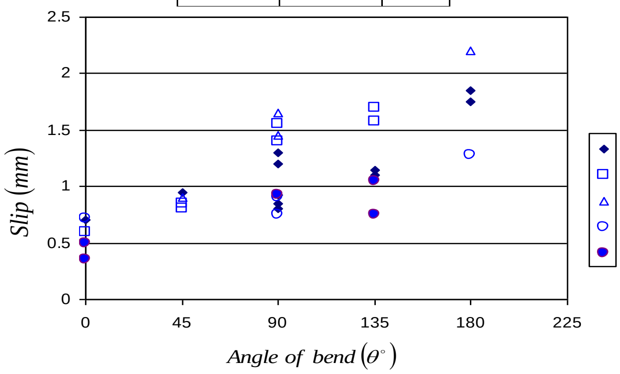
Reference (Minor J. *et al*. 1975) includes data on bar stresses at 0.25 mm lead-end slip. That for straight anchorages fits the equation used for Hribar and Vasko's tests satisfactory. For bent anchorages the scatter of results is very large but almost all the bar stresses are lower.



(a) Influence of (r/φ)

Key

φ (mm)	$\frac{l_b}{\varphi}$	θ (°)
16	7.2	90
22	7.3	90
22	4.9	45
22	4.9	90
22	9.7	90



(b)Influence of θ

Key

φ (mm)	$\frac{l_b}{\varphi}$	$\frac{r}{\varphi}$
16	9.4	2.4
22	7.2	2.4
22	7.2	1.6
22	9.7	2.3
22	7.3	2.3

Fig. 5 Loaded end slips at $f_s = 414 N/mm^2$ - tests by Minor and Jirsa

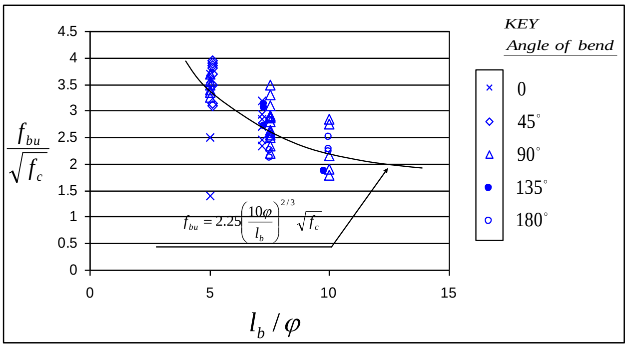


Fig. 6 Influence of bond length on bond strength in tests by Minor and Jirsa

2.5 Marques and Jirsa(Marques J.L.Get al. 1975)

Marques and Jirsa tested twenty-two specimens with bent bar anchorages in beam-column joints. The bar sizes were 22.2 mm and 34.9 mm and bend radii and tail lengths were all in accordance with ACI 318-71(ACI 318-71, 1971)

The principal variables in the series related to the lateral confinement of the bent bars within the joint. As shown in Fig.(7) they were the positions of the column bars relative to the anchored beam bars, the side cover and the transverse reinforcement.There were also variations of the axial load on the column and of the bonded lead length before the start of the bends. The bars were generally bent through 90° but four tests were made with 180° hooks.Slips were measured at the column face and at locations within the column and strains were measured outside the column and at the starts and ends of bends.

In nearly all the tests the first cracking was on the inside face of the column and the cracks radiated from the loaded bars. This was followed by vertical cracking on the side faces in line with the column bars near the inside face and cracking along the lines of the bends. Failure occurred with large areas of the side faces spalling away. Column bars outside the bent bars and the side face legs of stirrups were bent outward.

The slip measurements showed that most of the movement was in the lead length and bend. Strains were similarly concentrated toward the loaded end although the values at the ends of the bends increased rapidly as failure approached. The test data are summarised in table (3).

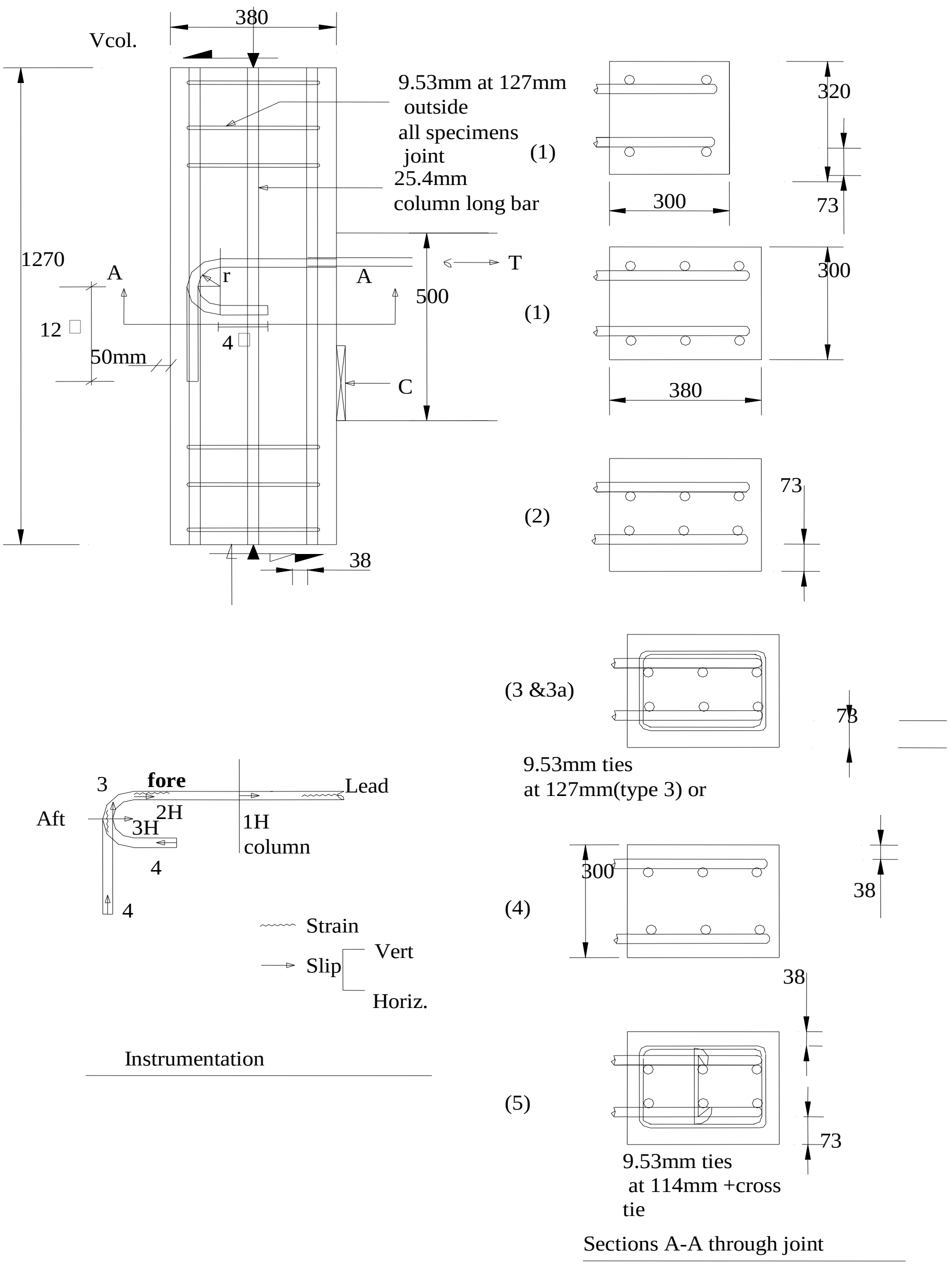


Fig. 7 Marques and Jirsa's test arrangements

Table 3 Summary of data for tests by Marques and Jirsa

Spec. no	$\frac{c_s}{\varphi}$	Links(2) in joint	Pos.(3) of col. bars	$\frac{l_1}{\varphi}$ (4)	f_c N/mm^2	σ_c (5) N/mm^2	Bar stresses at lead slips of…. (mm)			Δ_u (6) (mm)	f_{su} N/mm^2
							0.13	0.41	1.27		
Bars φ 22.2 mm (7/8")											
1	3.29	-	out	10.9	31.7	21	227	379	531	3.8	627
2	3.29	-	out	10.9	34.8	10	241	421	538	4.6	689
3	3.29	-	out	10.9	29.6	6	207	358	538	5.3	669
4	3.29	-	out	7.4	28.6	20	138	214	407	2.0	434
5(1)	3.29	-	out	10.9	27.6	21	165	310	496	3.8	600
6(1)	3.29	-	out	7.4	30.0	20	159	262	407	1.8	421
7	3.29	-	in	10.9	32.8	21	227	358	517	6.4	683
8	3.29	-	in	10.9	32.8	10	221	317	483	5.3	655
9	3.29	(a)	in	10.9	32.1	21	255	448	552	5.3	717
10	3.29	(b)	in	10.9	25.9	20	255	434	552	5.6	676
11	1.71	-	in	10.9	31.0	21	200	372	503	1.3	510

Bars $\varphi\,34.9\,mm$ ($1\frac{3}{8}''$)											
12	2.09	-	out	4.4	33.8	21	131	227	331	1.5	338
13	2.09	-	out	4.4	32.8	6	131	200	331	1.5	358
14	2.09	-	out	2.2	31.7	21	124	207	-	1.0	290
15(1)	2.09	-	out	4.4	30.3	21	90	172	296	1.3	303
16(1)	2.09	-	out	4.4	30.0	5	179	255	-	1.3	345
17	2.09	-	in	4.4	34.5	21	152	241	331	1.5	331
18	2.09	-	in	4.4	31.0	5	124	193	345	1.5	365
19	2.09	(a)	in	4.4	33.4	6	110	193	365	2.3	427
20	2.09	(b)	in	4.4	34.5	7	165	290	455	1.5	476
21	1.09	-	in	4.4	28.3	5	179	269	-	0.8	303
22	2.09	(c)	in	4.4	34.5	5	159	269	434	2.0	455

Note:
1.Specimens with 180° hooks, all others had 90° bends.
2.(a)- $\varphi 9.5$ @127 , (b)- $\varphi 9.5$ @63 , (c)- $\varphi 9.5$ +cross tie @63
3.Out-column bars outside beam bars , In- column bars inside beam bars.
4. l_1 =lead length.
5.Column load/column area.
6.Lead slip at failure.
For $22.2mm$ bars r/φ =3 ; for $34.9mm$ bars r/φ =4
For 90° bends tail length= 12φ ; for 180° bends tail length= 4φ
f_y for the anchored bars =450 to 475 N/mm^2

The conclusions reached by Marques and Jirsa were

1. In contrast to the results from Minor and Jirsa, there was little difference in behaviour between the specimens with 90^o and 180^o and bends.
2. The specimens with longer lead lengths developed less slip and higher bar stresses than those with shorter leads.
3. The location of the main column bars had little effect.
4. Column links within the joints improved performance, particularly for the larger beam bars.
5. Axial loading of the column had little effect.

The relevant provisions of ACI 318-71 were reviewed in the light of the test results. The bond in the lead length was calculated via the basic expression for development lengths for bottom-cast bars. The bar stresses developed by bends plus tails were calculated as $\xi\sqrt{f_c}$ where $\xi = 14.8$ for $\varphi = 22.2$ and $\xi = 29.9$ for $\varphi = 34.9$ (ξ in SI units).The ratios of experimental to computed bar stresses varied from 1.19 to 1.86 with a mean of 1.47 and a coefficient of variation of 13.3%.

An alternative design procedure was proposed. According to this the bar stress developed by a standard hook or bend is

$$f_h = 58(1-\varphi/85)\psi\sqrt{f_c}\ \ (\mathrm{N}, mm) \tag{3}$$

Where: ψ =1.4 if $\varphi < 35mm$ and the bonded lead length $l_1 \geq 102mm$ and the side cover is not less than 63*mm* and the cover on the tail extension is not less than 51 *mm*. ψ =1.8 if the conditions above are complied with and in addition the joint is confined by closed ties at a spacing $\leq 3\varphi$. ψ =1.0 in all other cases.

No increase in bar stress is allowed for tail extensions or bend radii greater than those of standard hooks and bends. If the bonded lead length $\succ 102mm$, the increase of bar stress can be calculated assuming a bond stress equal to $8\sqrt{f_c}/\varphi$. In overall terms this modification of ACI 318 has little effect on the correlation of experimental and computed strengths. The mean ratio is reduced from 1.47 to 1.43 while the coefficient of variation is unchanged.

The Marques and Jirsa alternative is clearly the basis of the current ACI 318.This does, to some extent, explain the oddly arbitrary use of fixed covers etc. rather than dimensionless quantities such as c_s/φ . Further analysis of these results would be very difficult. Inspite of strain measurements having been made at the beginnings and ends of the bends, the results are given for only two specimens, one with 22.2 *mm* bars and one with 34.9 *mm* bars. In both cases, the tail bond stresses were initially much lower than those in the curves, but were rising rapidly close to failure. For the smaller bars, the lead length bond stresses were initially highest , but reduced in the latter stages, while for the larger bars they were initially small and were practically zero above half the ultimate load. With the forces at the starts of the bends unknown it is unrealistic to make comparisons with calculated bearing resistances. If the bond stress is taken as being uniform along the whole length l_b the bearing stresses for the specimens without transverse reinforcement are on average 27% above BS8110 predictions for the smaller bars but for the large bars they are 24% below BS8110 values. The variation may be a matter of behaviour in relation to bond/anchorage but there are also issues related to the other actions in beam-column joints. As can be seen from table (3) the provision of stirrups within the joint region did increase the forces anchored but for the smaller main bars the stirrups at S_t=5.7 φ gave a higher result than that for stirrups at 2.85φ . For the larger bars, with $s_t = 1.8\varphi$ the results were higher than for stirrups at 3.6φ

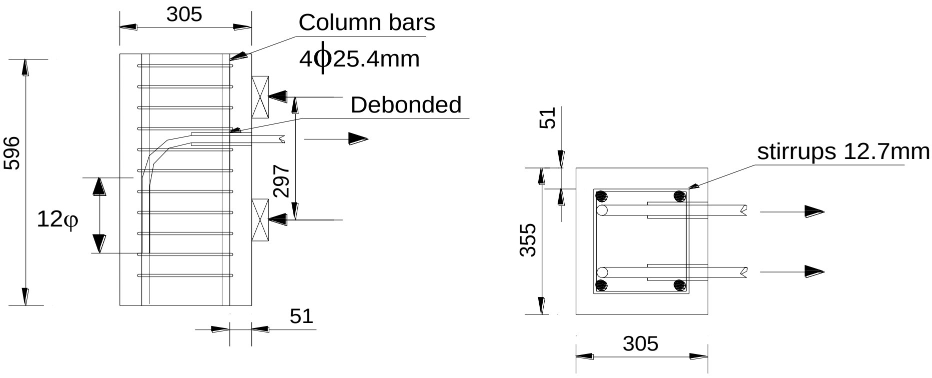


Fig. 8 Soroushian *et al*'s test arrangements

Table 4 Summary of results of tests by Soroushian *et al*.

Spec. no.	f_c $\left(\frac{N}{mm^2}\right)$	φ (mm)	Stirrups	$\frac{A_{st}}{s_t\varphi}$	F_{su} kN	f_{su} N/mm^2	$\frac{f_{bu}}{\sqrt{f_c}}$	σ_{bu} N/mm^2	$\frac{\sigma_{bu}}{f_c}$
1	26.1	25.4	φ 9.5@76.2	0.037	294	580	1.62	152	5.82
2	26.1	25.4	φ 9.5@76.2	0.037	268	528	1.48	138	5.31
3	26.1	25.4	φ 9.5@101.6	0.028	208	410	1.15	107	4.12
4	26.1	25.4	φ 12.7@76.2	0.065	313	617	1.72	162	6.20
5	41.7	25.4	φ 9.5@76.2	0.037	292	576	1.27	151	3.62
6	26.1	19.1	φ 9.5@76.2	0.049	193	674	1.88	176	6.75
7	26.1	31.8	φ 9.5@76.2	0.029	350	441	1.13	87	3.31

Note: specified yield stress of reinforcement= 480 (N/mm^2), F_{su} = force in one bar.

2.6 Soroushian , Obaseki , Nagi and Rojas(Soroushian P. et al. 1988)

Soroushian*et al*. tested anchorages of 90^0 bends. The test arrangements shown in Fig.(8) to some extent simulated conditions at beam-column joints, but without vertical load and with unrealistic reactions on the columns. In all the tests the bends had 12φ long tails and ACI standard internal radii of 3φ for $19mm$ and $25mm$ bars and 4φ for $32mm$ bars. The variables were the confining reinforcement, the concrete strength and the bar size. Behaviour was similar in all tests. Initially at about half the ultimate load horizontal cracking occurred on the side faces in the plane of the two bars, and extended along the bends with increasing load. Vertical cracks in the planes of the bends appeared close to failure and the side cover split away. The test results are summarised in table (4) for which ultimate loads have been scaled from graphs in (Soroushian P. *et al*. 1988). Soroushian *et al*. conclude that

1. Pull-out resistance increases with increasing bar diameter , but the increase is lower than the corresponding rise in the bar yield force. (The ultimate bar force was approximately proportional to φ in the three relevant tests).
2. Confinement of concrete surrounding the hook is an important factor influencing the hook performance.
3. Concrete compression strength did not influence the hook pull-out behaviour in the limited tests.

They also modified an empirical equation by Eligehausen for the relationship between the bar stress and the loaded-end slip to fit their test results. For the rising branch of loading, the expression given amounts to $F_{s,\Delta} = F_{s,u}(\Delta/2.54)^{0.2}$, with Δ in *mm* and $F_{s,u} = 271(0.05\varphi - 0.25)$ in *KN* and *mm* (Note the US to SI conversion in reference (55) is incorrect).

Conclusions 1 to 3 are a fair reflection of the test results. The third of them is the most surprising and is supported by only one variation of the concrete strength. The only obvious explanation for it would be that at the ultimate limit the restraint against splitting provided by the transverse reinforcement predominates over that from the concrete.

The second conclusion is supported by the results for four tests, all with the same bar size, bend radius, cover and concrete strength. The circle points in Fig.(9) show σ_{bu}/f_c and A_{st}/s_t for these tests and do suggest a relationship passing through the origin. The two other points show that any general relationship would need to take account of the effects of bar size, and other factors. Account may also need to be taken of the presence of the large volume of concrete in front of the anchorages and not subjected to any direct effects from bond stresses. In regard to the first conclusion, it can be noted that, in the relevant tests, the forces developed were approximately proportional to the bar size, but the tests did involve variations of the ratios c_s/φ and r/φ as well as the ratio of the area of transverse reinforcement to that of the anchored bars.

In relation to serviceability the proposal by Soroushian applied to the typical specimens with 25.4 *mm* bars gives a steel stress of 233N/m^2 at a slip of0.25 *mm* and this agrees well with 247 N/m^2 given by the equation derived above from the tests by Hribar and Vasko.

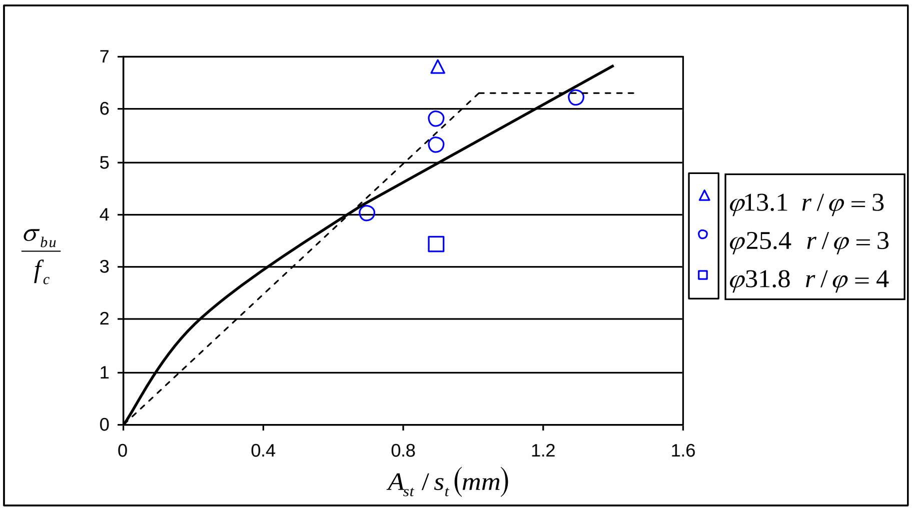


Fig. 9 Influence of transverse reinforcement on ultimate strength -tests by Soroushian *et al*.

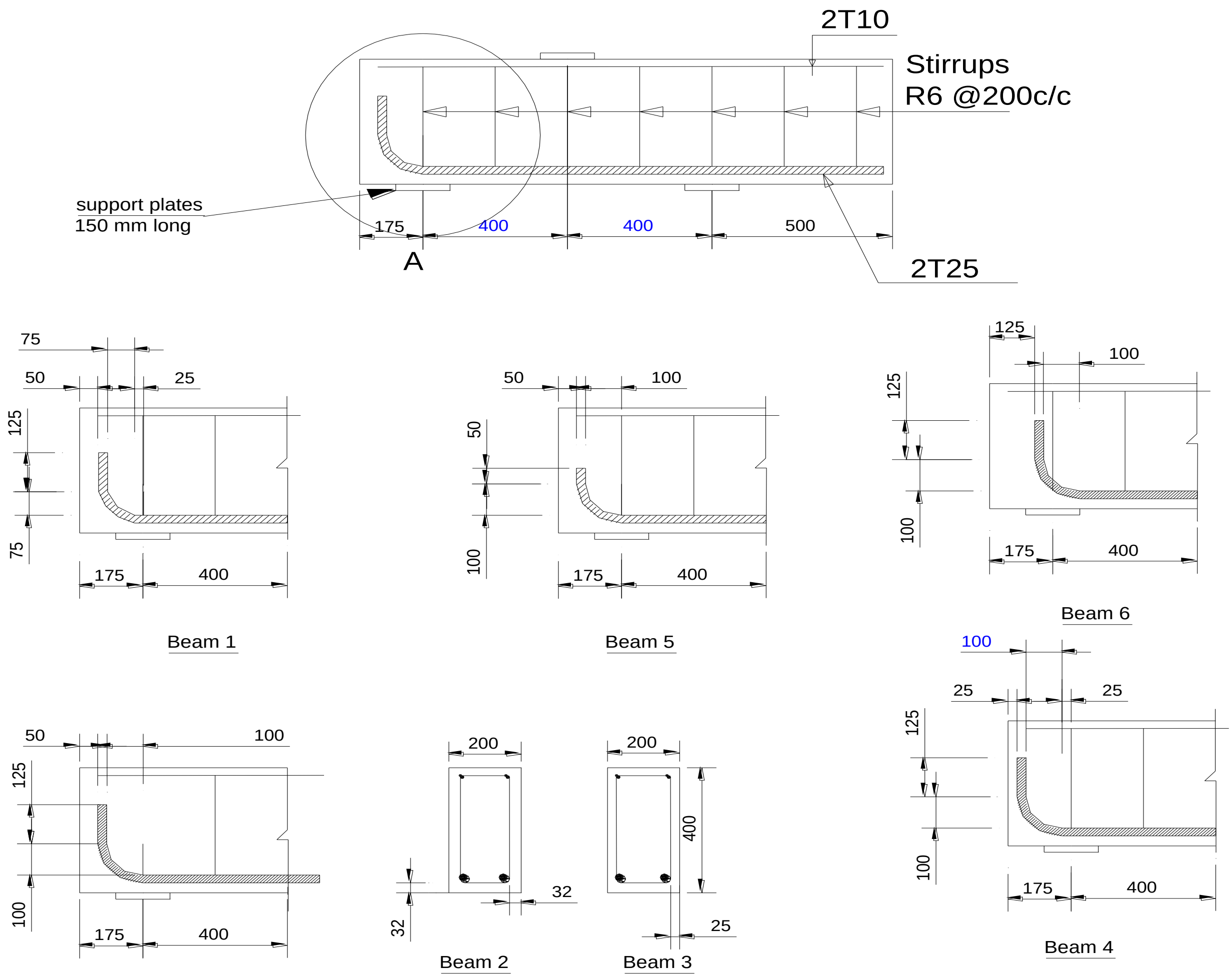


Fig.10 Detailing of beams by Gulparvar

Table 5 Ultimate bond stresses at anchorages (N/m^2)for Gulparvar's beams G4 and G6

Beam No	Average	Lead length	Bend	Tail
G4	8.00	8.10	12.16	2.03
G6	8.84	-	$< 7.92^*$	6.50

Table 6 Results of tests by Gulparvar

Beam No.	l_b mm	l_1 mm	c_e (4) mm	f_{cu} $\left(\frac{N}{mm^2}\right)$	P $\left(\frac{N}{mm^2}\right)$	F_{bu} (kN)	F_{RK1} (kN)	F_{RK2} (kN)	$\frac{F_{bo}}{F_{RK}}$
G1(1)	362.5	100	50	44.5	14.58	248	222	<u>195</u>	1.27
G2	376.7	75	50	53.4	15.00	252	<u>233</u>	264	1.08
G3(2)	376.7	75	50	51.0	14.83	249	247	<u>241</u>	1.03
G4(3)	401.7	100	25	53.2	15.00	>252	<u>269</u>	280	>0.94
G5	301.7	75	50	53.1	13.33	224	<u>202</u>	263	1.11
G6	301.7	0	125	51.0	12.50	209	<u>198</u>	204	1.06

Note: (1) $r = 75mm$ in beam G1, $100mm$ in all others
(2) $c_s = 25mm$ in beam G3 , $32mm$ in all others
(3) beam G4 failed in shear in the other shear span
(4) c_e = end cover
(5) in columns 8 and 9 underlinings indicate the critical calculated strength

3. Simply supported beam experimental work by Gulparvar(Gulparvar D., 1997)

Gulparvar tested six simply supported beams to investigate the anchorage capacities of 90^0 bends at supports. Details of the beams are shown in Fig.(10). The variables were the tail length, the radius of bend, the side cover, the end face cover and the straight lead length over the support. In all but one of the tests failure occurred at the support with the bent anchorages. In these cases there was generally cracking on the side faces along the lines of the bars followed by vertical cracking (splitting) visible on the end face in line with one or both of the bars. In beam G1 there was a horizontal crack on the end face at the level of the tops of the bars. In beam G5, where the tail lengths were short, the failure seemed to be purely by the bars pulling around the bends as there was no cracking on the end of the beam. In beam G6, where the bends started at the inner face of the support, the concrete within the bends spalled out sideways. Beam G4 failed by shear in the shear span with straight anchorages. The bar stresses at midspan were at or close to yield ($f_y = 480-520N/mm^2$) but those at the supports were lower due to the action of the single stirrup ($f_y = 260N/mm^2$) in the shear span.

In beams G4 and G6 there were strain gauges at opposite ends of diameters at the beginnings an ends of the bends, and there was an extra pair at midspan in beam G6. Bond stresses calculated from these strains are given in the table (5).

For the normal anchorage in beam G4 with the bend starting just beyond the centre of the support, the bond in the tail remained low, although it should be noted that there was no anchorage failure in this beam and the tail length might have become more active in an anchorage failure. In beam G6, where the bend began at the inner face of the support and the anchorage length was reduced the tail bond stress was much closer to the average.

Gulparvar proposed that the characteristic anchorage capacity of a bar could be calculated as the lesser of the values corresponding to bond and bearing failures.

$$F_{RKb} = f_{bp}\pi\varphi\, l_b \quad (4)$$

$$F_{RK\sigma} = f_{bp}\pi\varphi\, l_1 + \sigma_b r\varphi \quad (5)$$

Where f_{bp} is the BS8110(BS8110,2005) bond stress ($f_b = 0.7\sqrt{f_{cu}}$) modified by the MC90/EC2 (EuroCode 2,2004)coefficient allowing for the effect of transverse pressure, $f_{bp} = f_b/(1-0.04p) \le 1.67 f_b$ with P in Nm^2 (note: the ratio $f_{bp,\max}/f_b$ is greater than in EC2, but for the beams in question $f_{bp,\max}$ is practically as in EC2 due to the Eurocode's higher value for f_b). The upper limit governs for all of Gulparvar's beams, P is the transverse pressure at the support, l_b is the bonded length (lead+bend+tail) over the support, l_1 is the lead length over the support and σ_b is the limit stress for bearing in the bend as given by BS8110.

Table (6) summarises the results of a comparison between the actual anchorage resistances and values calculated as above.

The agreement between the calculated and experimental resistances is good, but the range of tests is very limited.

The application of the influence of p to the whole anchorage length including the vertical tail section seems rather dubious.

4. Theoretical approach by Schiessl(Schiessl P.,1982)

Schiesslstudied anchorage resistance as a combination of components from straight bar lengths and terminal bends or hooks(or welded transverse bars). Bond – slip relationships for straight bars were derived from the results of pull-out tests with 5φ bond lengths and relationships between loaded-end bar stresses (f_{so}) and slips were found from tests of bends and hooks with 2φ bonded lead lengths as shown in Fig.(11), (see Muller in 2.6.2).

$$f_b = f_{cu}\left(a + b\Delta^c\right) \quad \text{- for straight lengths} \quad (6)$$

$$f_{so} = f_{cu} d\Delta^e \quad \text{- for bent anchorages} \quad (7)$$

where f_b = bond stress f_{so} = steel stress, 2φ before start of a bend.

Δ =slip in general for straight bars or loaded-end slip for bent anchorages.

The values of the coefficients *a* to *e* depend upon the bar size and the bars's position during concreting.

$$\frac{d\Delta}{dx}=\frac{f_s}{E_s}\left(1+\frac{f_c}{f_s}\frac{E_s}{E_c}\right) \quad (8)$$

$$\frac{df_s}{dx}=\frac{4}{\varphi}f_b \quad (9)$$

Where f_s = bar stress at section *x* and f_c= concrete stress at section *x*. f_b = bond stress at section *x*.

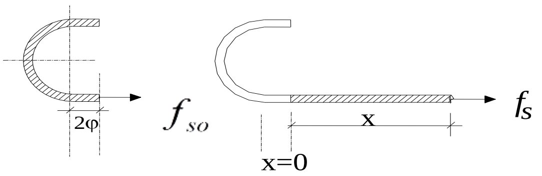


Fig. 11 Bond lengths for bent and straight parts of anchorages in Schiessl's approach

If conditions at one section e.g. at the loaded end of a hook+ 2φ, are specified, the development of stress along the straight length of the bar can be found from equations (6, 8 and 9) and Schiessl used a finite difference approach for this. The calculations start with defining a straight anchorage length from 2φ in front of bend then:

1- f_{so} can be calculated for any assumed Δ using equation (8)

2- Calculating f_b for the value of Δ by eqn.(6) and assuming it applies along a defined length,then at the end of the length the developed f_s is sum of the increment calculated from eqn. (9) and $f_{s,0}$ at the start of the length.

3- The developed elongation can be calculated from the average stress in the length, and Δ at the end of the length is the sum of this and the slip 2φ from the bend.4- The process can be repeated for subsequent lengths using end values of f_s and Δ for one length as the start values for the next length till the developed steel stress reach the required working value of 24*N/m*^{2}.

Schiessl's criterion for determining the development length for a bar with an end hook was that the slip at the section of maximum bar stress should not be significantly greater than that for a straight development length.

The maximum bar stress considered was related to working load design and was 240 N/mm^2. With this level of maximum stress the slip at the start of the bend is relatively small and the contribution of the hook is limited. Fig.(12), which is reproduced from (Schiessl P.,1982)shows the reduction of anchorage length which is obtained as a function of the increase of slip at the beginning of the anchorage.The 30% reduction used in DN1045 and the CEB-FIP model code of 1978(and subsequently in EC2) corresponds to a 5 to 10% increase of slip.

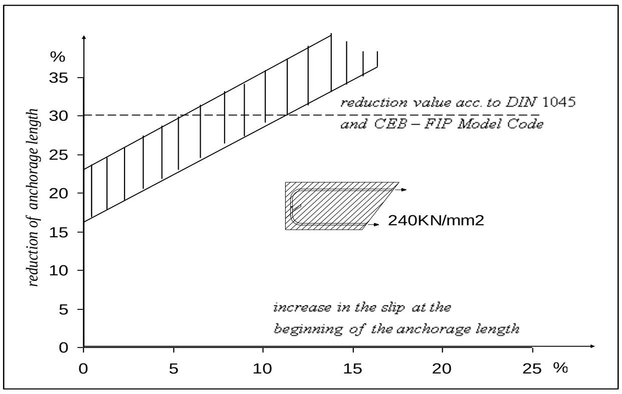


Fig. 12 Results of all calculations $\varphi = 8-28mm$, $f_{cu}=25-55N/mm^2$ for ribbed bars in upper and lower positions from Schiessl

This work is of interest as it appears to be the background of EC2's treatment of hooks and bends using the coefficient α_1. It does however have severe limitations. It is a working stress method and not one for the ultimate limit state. Its criterion for determining the deformation limit, which governs the contribution of a hook, is not a realistic performance criterion as it does not define capacity or even a crack width. As applied by Schiessl it has the further problem that the concrete stress f_c, and thence the shortening of the concrete, is determined assuming that a prism of concrete develops a stress equal to $f_c(A_c/A_s)$,where A_c is the area of the prism and A_s is the area of the bar. This is not realistic for the flexural tension zone of a beam, in which the concrete cracks. Using the expression $f_{so}=f_{cu}d\Delta^e$ to calculate the bar stress at a slip of 0.255*mm*,with *d*=16.5 for an upward bend and *e*=0.47, both as given by Muller, for $f_c=30N/mm^2$ ($f_{cu,200}$ as presumably used by Muller $\approx 33.75N/mm^2$), $f_{so}=290N/mm^2$ which agrees exactly with the expression from Hribar and Vasko's results for a standard $r=2.5\varphi,\ l_t=2.5\varphi$ 180^o bend with a 2φ lead length.

Conclusions

The only one of the works reviewed that applies directly to anchorages at simple supports isGulparvar's dissertation. His proposal to limit the ultimate bond stress throughout an anchorage to a straight bar value (BS8110's characteristic stressenhanced to allow for the effect of transverse pressure) and to limit the bearing stress to BS8110's characteristic value agrees with his test data. The data are however very limited. He does not treat serviceability as such but plots maximum crack widths. At half of the full flexural capacity these never exceeded $0.3mm$, and the maximum widths were probably those of the flexural cracks at midspan rather than of the shear cracks running to the supports.

The other papers are of less direct relevance. Mylrea's tests were made on plain round bars, but do show that the current treatment of bearing stresses which neglects the flexural stiffness of the bars in it's $\sigma_b = F_s/r\varphi$ leads to serious errors if r/φ is very small and may cause some errors at the present minimum $r/\varphi = 2$ .Schiessl's treatment of the compatibility of deformations between straight lead lengths and terminal bends and hooks is not of much relevance to conditions at a support if it is assumed that shear cracking may reach to the support although, it could be of interest for cases where there are no shear cracks and the location of the flexural crack closest to the support can be predicted.

Muller's work with pull-out tests, made on hooks (and some bends) in cube specimens, is limited to service load behaviour and most of the results are given only in terms of relative performance. His predictions of steel stresses as functions of loaded-end slip give results similar to those from the expression derived here from Hribar and Vasko's results for the bar stress at a slip of $0.25mm$.

A possibly important conclusion in his work is the considerable difference between the performances of downturned and upturned hooks, with the former developing only 53% of the bar stresses developed by the latter at a slip of $0.1mm$ .Schiessl's expressions suggest that the difference reduces as the slip increases up to $0.5mm$, but it must be very uncertain whether it becomes negligible at failure, as assumed in ACI 318.

The tests by Hribar and Vasko provide information on bond in relation to loaded-end slip and, to a lesser extent ultimate strength, in circumstances of effectively infinite cover in both directions normal to the straight part of a bar. The information on bond v loaded end slip should be of practical use with normal covers provided there is no splitting of the cover at service loads. That on ultimate bond strength can only be regarded as an upper limit applicable where the covers are large.

Minor and Jirsa's test arrangements were rather suspect, but the ultimate bond stresses, for both bond failures without splitting and other rather than ill-defined failures in which the concrete " fractured", fit the same trend as those inHribar and Vasko's tests, i.e. $f_{bu} \approx 2.25(10\varphi/l)^{2/3}\sqrt{f_c}$ .The results for loaded end slips were very scattered but the bar stresses at slips of $0.25mm$ were generally lower than those from Hribar and Vasko.

With much longer tail lengths, (12φ) for 90^o bends, and specimens realistically representing beam-column joints, the tests by Marques and Jirsa all ended in splitting of the cover in the planes of bends. They show similar strengths for 90^o and180^o bends and are the basis for the very prescriptive rules of the present ACI 318. They are difficult to analyse rationally, largely because of the lack of information on the distribution of bar stresses along the anchorages and particularly the stresses at the starts of the curves. They show that transverse reinforcement can increase ultimate capacity, but the few tests with transverse reinforcement are insufficient to allow any relationship to be developed between the ultimate bar stress and the many parameters involved.

Soroushian *et al*'s tests representing beam-column joints were much poorer representations of reality than those of Marques and Jirsa. The number of tests was also very small. The results suggest that with relatively heavy transverse reinforcement it is the stirrups, rather than the concrete that provide most of the restraint against splitting. The bar stresses at loaded-end slips of $0.25mm$agree well Hribar and Vasko's results.

For given bar and cover dimensions and concrete strengths, the performances of terminal hooks and bends are likely to be very considerably affected by the stress fields in the surrounding concrete. For any particular type of application, relevant experimental data are needed for the formulation of design guidance. Even though results for other applications may have relevance, this cannot be known in the absence of data for the case in question. The present situation seems to be that there are 5 results available from tests to failure of 90^o bends at supports and none for 180^o bends. There are probably isolated results from tests in which unintended anchorage failures have occurred, but the total of information is clearly very small. In these circumstances it was decided that the priority for the present work was to conduct a reasonable number of tests of one type which should be a realistic representation of conditions at a simple support and at the same time be relatively simple to interpret e.g. the anchorage lengths and the forces on them should be clearly defined.There should be variations of bar size, cover, radius of bend and tail and lead lengths for both and bends. To keep the number of tests manageable it was decided not to investigate the effect of transverse reinforcement, or the orientation of the bends relative to the direction of concreting.

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