Esben Jonsson

Shear capasity of prestressed hollow-core slabs

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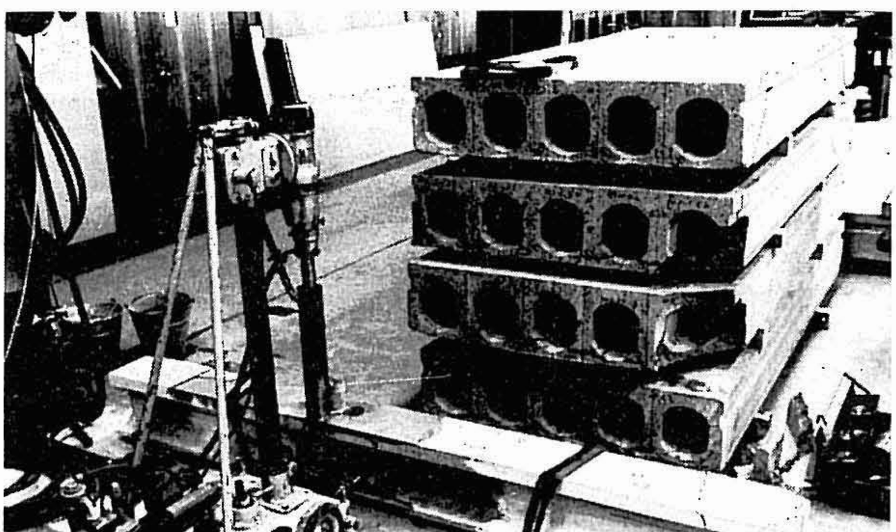

Shear capacity of prestressed hollow-core slabs

ESBEN JONSSON

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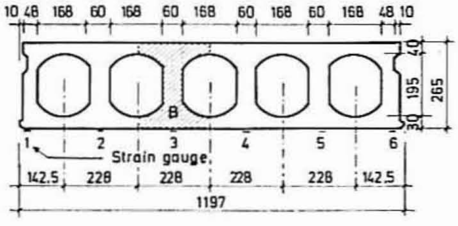
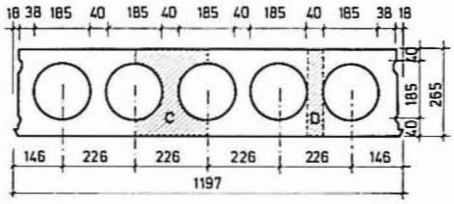
INTRODUCTION

This paper reports the results of a great number of full scale tests on factory-produced hollow-core slabs. The investigation was conducted at the Norwegian Building Research Institute. So far, very little research has been carried out to determine the shear capacity of the described type of slabs. Only few results from commissioned work ordered by industry in Europe and America are available.



Figur 1. Cross section of Series A and E where Series B is shaded. ■ Tverrsnitt av serie A og E hvor serie B er skravert.



Figur 2. Cross section of Series F and G approximate where Series C and D are shaded. ■ Tverrsnitt av serie F og tilnærmet G hvor serie C og D er skravert.

BACKGROUND

There are no special code provisions for hollow-core slabs as far as the author knows. Normally the calculations are done in accordance with the general code for slabs and beams. According to the Norwegian Code [1] the smallest design shear capacity, without shear reinforcement and having a strand area as in present tests, is given by:

$$V_d = f_v\,(bd + 75\,A_s) \qquad (1)$$

where: f_v = design shear stress (N/mm^2)
b = the width (mm)
A_s = total area of main reinforcement (mm^2)

In a hollow-core slab b has normally been taken equal to the sum of the minimum rib widths. The effect of the prestressing force is normally neglected. Eq. (1) is based on the same principles as those on which the Model Code [2] has been based. In connection with the use of Eq. (1) for hollow-core slabs the following problems are important:

1. How strong is the anchorage of the strand?
2. What is the influence of the concrete extruding?
3. What is the influence of the geometry?
4. What is the correct value of f_v?

The special method of production (extruding) of these slabs makes the assembling of anchorage reinforcement and stirrups difficult if not impossible. The only way to increase the capacity is therefore to fill the cores near the support. Unfortunately this method is rather cumbersome and expensive. Still, the filling near the support often has to be done, because Eq. (1) gives low capacity.

The question of control of hollow-core slabs is of a considerable interest. Control criteria proposed by Anderson and Anderson [7] for flexural bond seems useful for the control of the shear capacity.

THE OBJECTIVE

The main objective of the present work is to establish:

1. Design shear capacity
2. Behaviour and mechanical model
3. Quality control criteria

An experimental investigation was the most convenient because of the many variables, and because the anchorage capacity seemed to be difficult to calculate.

TEST PROGRAMME

Procedure

The following is axamined:
1. Dimensions
2. Free end slip
3. Concrete strength
4. Deflection
5. Strand deformation
6. Slab stresses
7. Cracks
8. Failure

Test specimens

These consist of hollow-core slabs and beams grouped into a total of 7 different series (A to G). The beams are parts of the slabs. The cross section of Series A and E (Dy-core or Spenndekk as named in Norway) is shown in Fig. 1. Series B is sawn off from the slab. Series F (Spiroll) is shown in Fig. 2. Series C and D are wet casted and the strands were not prestressed. The section of Series G (Elematic) is roughly the same as Series F (Spiroll), but the production equipment is different.

Seires A, E, F and G have the following data:

Section area:
$A_c = 173 \cdot 10^3\ mm^2$

Concrete:
extruded
cube strength 55 N/mm^2

Reinforcement:
7-wire strands
diameter 12,5 or 9,5 mm
02-limit 1750 N/mm^2
total aera A_s (bottom)

$(A_s/A_c)\ 100 = 0{,}23 - 0{,}58$

The properties of concrete are given in Tab. 1.

TABLE 1. Concrete properties
– Betongblanding

Materials		Series A, B and E	Series C, D and F	Series G
Sand,	mm	0 – 1	0 – 4	–
Gravel,	mm	0 – 8	0 – 8	0 – 8
Crushed rock	mm	4 – 8	8 – 12	8 – 16
Cement/aggregate (weight)		1 :5,6	1 :7,3	1 :5,0
W/C (weight)		0,32	0,32	0,29
Admixture		Betokem	–	–

Summary of tests

The test setup and equipment are shown in Fig. 3 and 4.

Series A, E, F and G: 55 tests
" B, C and D: 59 "
Variables (Fig. 3): s = 25 – 425 mm
a/d = 0,5 – 3
Age of slabs: 22 – 317 days

TEST RESULTS

Dimensions

The measured values differed very little from the nominal ones. The average distance d_s (Fig. 3) to the centre of the strands was 32 mm for Series A, 34 for E, 38 for F and 27 for G.

Free end slip

The values as shown in Fig. 5 are small in spite of the fact that many of the elements were considered to be of second class quality. However, the slips within the same element varied considerably. The results do not indicate that the free end slip increases by time.

Concrete strength

Four cylinders were core-drilled vertically from the top of every element. Two of these were used to examine the compressive strength, the remaining two were used to examine the splitting tensile strength. The compressive strength is large with one exception, but varies considerably within the same element as shown in Fig. 6.

The ratio between the splitting tensile strength and the compressive strength is shown in Fig. 7. For Series E the splitting tensile strength is approximately $0{,}73\sqrt{f'_c}$, with f'_c in N/mm².

Slab stresses

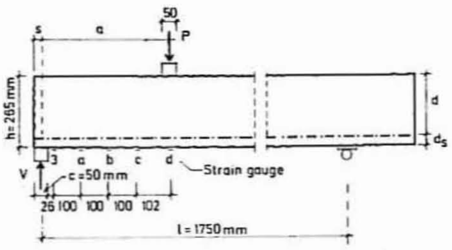
Strain gauges were mounted on only one element, as shown in Fig. 1 and 3. The concrete stresses indicate the effect of the prestressing and the load action, the results are shown in Fig. 8. The average of the compressive stresses from the prestressing force was 0,6 N/mm² in points 1 – 6 (Fig. 1). The ratio between the tensile stresses in points 3 and 3 d (Fig. 3) due to the load was:

0,21 with P = 50 kN
0,31 " " = 152 "
0,52 " " = 240 "
(P = 242 kN at failure)

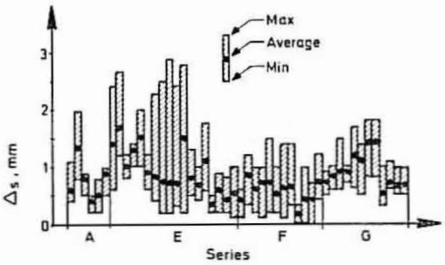
The stress was highest in the middle of the width of the slab.

Deflection

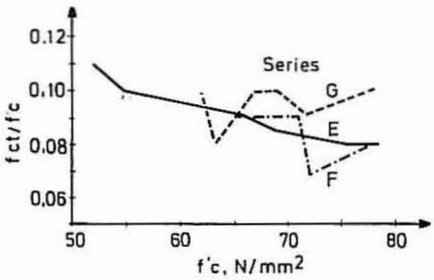
Except for a few tests there were no discontinuity in the deflection curves as shown in Fig. 9. This indicates that only few of the strands fail, if any, before the slab collapses.



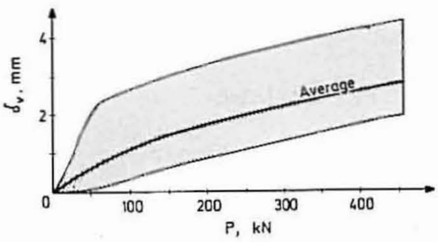
Figur 3. Test setup. ◘ Prøveoppstilling.



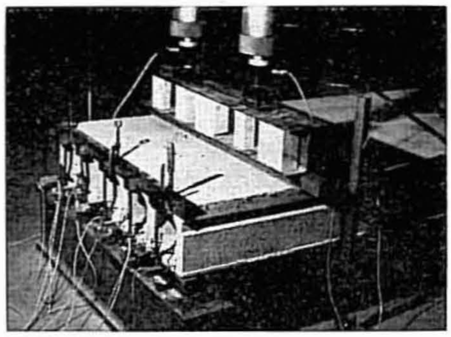
Figur 5. Free end slip Δ_s of strands in each test. ◘ Glipp Δ_s i spenntau før belastning.



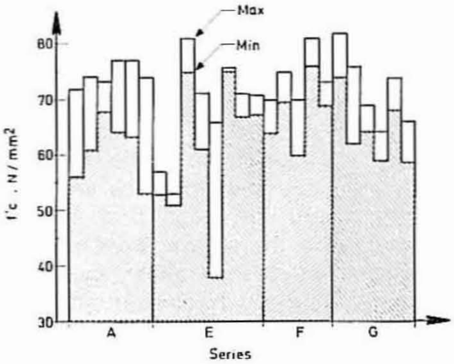
Figur 7. Ratio between splitting tensile strength f'_{ct} and compressive strength f'_c of concrete cylinders (73 x 73 mm). ◘ Forholdet mellom betongens spaltestrekkstyrke f'_{ct} og trykkstyrke f'_c for sylindre (73 x 73 mm).



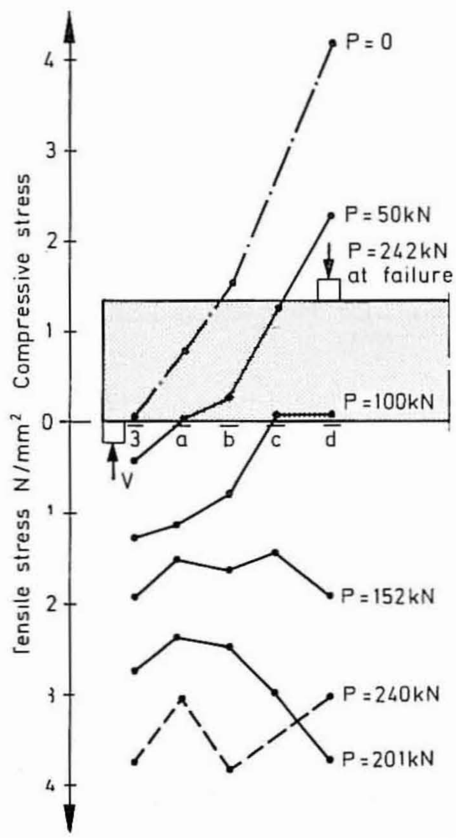
Figur 9. Relationship between deflection δ_v and load P for Series E, F and G. ◘ Forholdet mellom nedbøyningen δ_v og lasten P for serie E, F og G.



Figur 4. Test in progress. ◘ Prøvebelastning.



Figur 6. Compressive concrete strength f'_c as measured on two cylinders (73 x 73 mm) from each slab. ◘ Betongens trykkstyrke f'_c målt på 2 sylindre (73 x 73 mm) for hver plate.



Figur 8. Total stress at bottom of one slab with A_s = 400 mm² of Series E. ◘ Samlet spenning i underkant av en plate med A_s = 400 mm² fra serie E.

Cracks

The observed cracks are caused by:

1. Diagonal splitting
2. Shear bending
3. Anchorage tension
4. Flexural tension
5. Shear
6. Diagonal compression
7. Instability of the arch

The most frequent type of cracks are shown in Fig. 10 – 12. The ratio between the load P at the first visually observed crack and at failure was 0,8 – 1,0. The distance c from the edge of the support to the end of the element was in this case 50 – 100 mm.

If c is small, it is not necessary to calculate the crack width. If c > 400 mm the crack width must be controlled.

Flexual tension will cause cracks when the area of strands A_s is small. Shear may cause horizontal cracks between the pressure flange and the rib (especially the outer rib) if the compaction of the concrete is bad and/or the width is small. Diagonal compression and instability of the arch can only take place when both A_s and c are especially large.

Shear capacity at failure

Obviously both the geometry and the reinforcement are important, as shown in Table 2. It is assumed that the geometry of Series B and C cause about the same capacity. Based on Series B1 and C, as shown in Table 2, the shear capacity increases 31% because of the prestressing force.

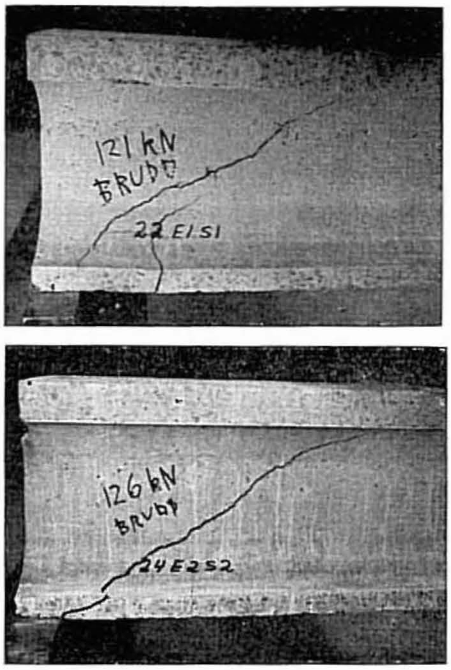
When the distance c is small the shear capacities are not much affected by the ratio of shear span a to effective dapth d as shown in Fig. 13. It is assumed that a/d = 2, s = 25 mm and the support length 50 mm provide the smallest shear capacity. When s increases, the shear capacity increases too as shown in Table 3.

The ultimate shear capacities are shown in Fig. 14. The characteristic shear capacities V_{ko} for each series are given in Fig. 15. There is a probability of 25 % that more than 10 % of the slabs will have a lower value than V_{ko}.

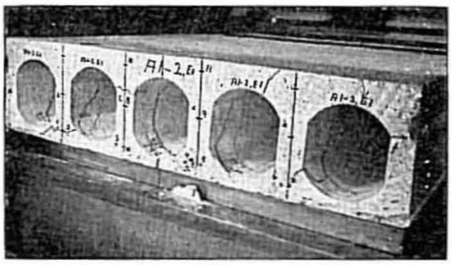
TABLE 2. Comparison of average shear capacity V_{um} at failure for Series B, C and D with a/d = 1,5 and s = 125 mm.
– Sammenligning av midlere skjærkapasitet V_{um} ved brudd for serie B, C og D med a/d = 1,5 og s = 125 mm.

Series	n	A_s mm²	V_{um} kN	R
D	6	100	36	C/D = 1,36
C	6	"	49	B1/C = 1,31
B0	2	0	24	B1/D = 1,78
B1	6	100	64	B1/BO = 2,67
B2	2	200	95	B2/B1 = 1,48

n = number of tests
R = ratio of capacities



Figur 10. Typical cracks at failure for Series B. ■ Typiske riss ved brudd for serie B.



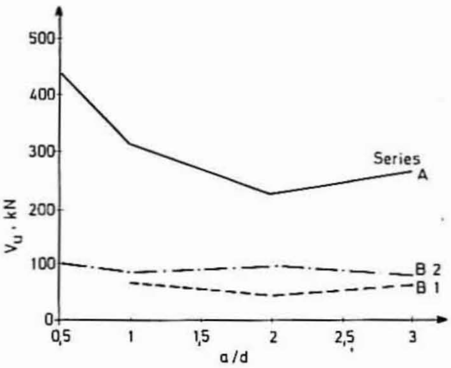
Figur 11. Typical cracks at failure for Series A, E, F and G. ■ Typiske riss ved brudd for serie A, E, F og G.

Figur 12. Cracks at the end of the strand after failure. ■ Riss i enden av spenntauet etter brudd.

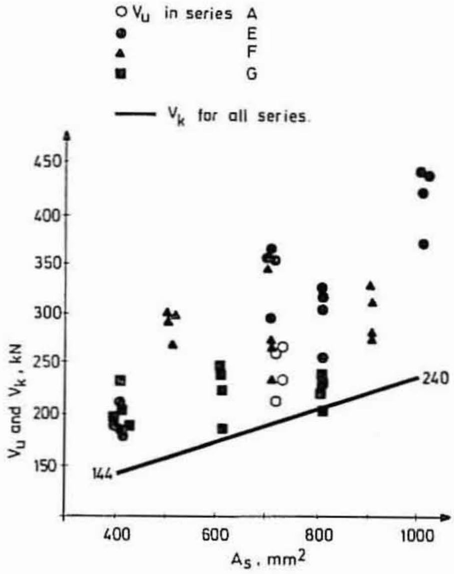
TABLE 3. Relationship between average shear capacity V_{um} at failure and distance s for Series B2 with a/d = 1.
– Forhold mellom midlere skjærkapasitet V_{um} ved brudd og avstanden s for serie B2 med a/d = 1.

s mm	n	V_{um} kN	%
25	4	88	100
125	2	99	113
225	2	123	140
425	2	188	214

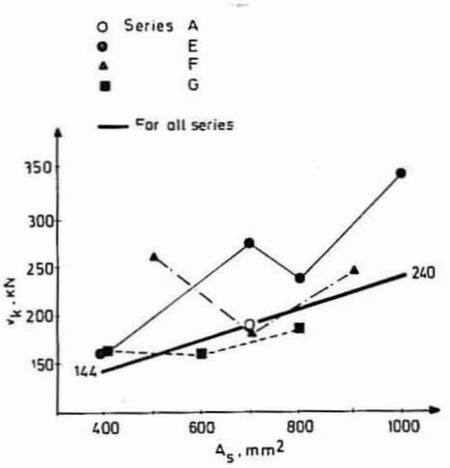
n = number of tests



Figur 13. Relationship between average shear capacity V_{um} at failure and a/d for Series A and B with s = 25 mm. ■ Forholdet mellom midlere skjærkapasitet V_{um} ved brudd og a/d for serie A og B med s = 25 mm.



Figur 14. Ultimate shear capacity V_u and common characteristic capacity V_k for Series A, E, F and G where: h = 265 mm, a/d = 2 and s = 25 mm. ■ Skjærkapasitet V_u ved brudd og felles karakteristisk kapasitet V_k for serie A, E, F og G med h = 265 mm, a/d = 2 og s = 25 mm.



Figur 15. Characteristic capacity V_{ko} for each series and common characteristic capacity for V_k for Series A, E, F and G. ■ Karakteristisk kapasitet V_{ko} for hver serie og felles karakteristisk kapasitet V_k for serie A, E, F og G.

The common capacity V_k is estimated as a straight line with a reasonable degree of safety and given by the formula:

$$V_k = 80 \cdot 10^3 + 160\ A_s \qquad (2)$$

where: V_k = characteristic shear capacity (Newton)
$400 \leq A_s \leq 1000\ m^2$

The first term in Eq. (2) is the concrete contribution, while the second is from the strands. The influence on V_u of the concrete compressive strength f_c between 50 and 80 N/mm² is rather small, as shown in Fig. 16. When f_c is lower than 40 N/mm² the influence may be considerable, as shown by Regan [6]. According to FIP [4] and to Anderson and Anderson [7], the anchorage capacity is not much influenced by f_c. Important is good consolidation of concrete around the strands.

It should be possible to expand Eq. (2) into a general formula following the same principle used in the Model Code [2], depending on:

1. Shape of cross-section
2. Effective depth
3. Rib width
4. Concrete strength
5. Area of strands
6. Prestressing

But it is doubtful to determine a general formula on basis of these tests, because the range of the variables in points 1 – 3 is too small and the concrete strength is too high.

BEHAVIOUR AND MECHANICAL MODEL

Before failure, the element carries the load through a combination of beam- and arch action. Diagonal tension and shear bending introduce the failure.

The mechanical model is shown in Fig. 17. The tension force F_t consists of the concrete force F_{ct} and/or the steel force F_s, depending on when and where the cracks occur. Hewover, hollow-core slabs with a short support at the end of the element collapse because of anchorage failure. If the diagonal crack comes simultaneously or after the shear bending crack, the diagonal splitting force may work together with F_t, otherwise not. The width of the rib may also influence the shear capacity. But the slab does not collapse because of diagonal splitting as long as the shear bending and the anchorage capacity are larger. The dowel action is probably not important, because A_s/A_c is small.

DESIGN SHEAR CAPACITY

Line load

On basis of the test results it is proposed that the design capacity is given by:

$$V_d = k_1\ k_2\ k_3\ V_k = 0{,}55\ V_k \qquad (3)$$

where: k_1 = 0,91 according to NS 3473 [1]
k_2 = 0,80 because of unknown distribution of load
k_3 = 0,75 because of unexpected impact load

The factors k_2 and k_3 are both conservatively estimated. A long term load reduces V_d probably less than 25 % if the concrete splitting stress, or the concrete compressive strength causes the failure.

The design shear capacity in Newton for one slab is given by Eqs. (2) and (3):

$$\boxed{V_d = 0{,}55\ (80 \cdot 10^3 + 160\ A_s)} \qquad (4)$$

where: $400 \leq A_s \leq 1000\ mm^2$
and: V_d = 79 – 132 kN

Uniformly distributed load

The shear capacities are similar for a line load and an uniformly distributed load when a/d = 2 according to Leonhardt [5]. Because the load distribution can be uneven, it seems to be adequate to use k_2 = 0,80 in this case too. Consequently the design shear force can be calculated by Eq. (4) which gives about twice as large capacity as Eq. (1).

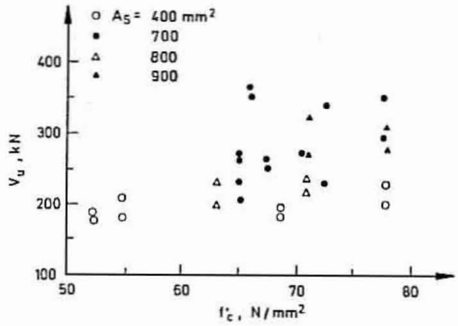
Fire resistance

This depends on the anchorage capacity when the length c is short. With the cover d_s like 35 mm and fire for a periode of 90 minutes, the following was found in the literature:

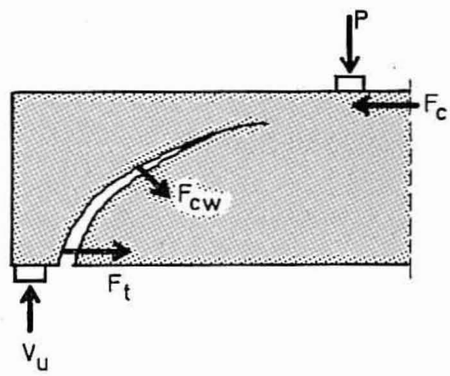
1. Temperature in the strands [10] = 490°C
2. Concrete compressive strength [9] 60 % *
3. Anchorage capacity [11]
   – round steel = 48 % *
   (f_c = 35 N/mm²)
   – shaped steel = 80 % *

* 100 % at 20°C.

The temperature 490°C corresponds very well with the temperature measured on Dy-core and Spiroll by Underwriters Laboratories in USA. On the other hand the temperature in the strands above the support may be lower if the support length is larger than 50 mm.



Figur 16. Relationship between ultimate shear capacity V_u and concrete compressive strength f'_c for Series A, E, F and G.
■ Forholdet mellom skjærkapasitet V_u ved brudd og betongens trykkstyrke f'_c for serie A, E, F og G.



Figur 17. Probable mechanical model.
■ Sannsynlig statisk modell.

The anchorage capacity of strands in tests similar to the tests by Reichel [11] would probably be 50 – 60 % of the capacity without fire. According to FIP/CEB [9] concrete splitting tensile strength is 55 % when the temperature is 490°C. However, these two references are dealing with wet casted concrete. It appears reasonable to assume that the anchorage capacity for hollow-core slabs as described in this paper, is at least 0,75 · 55 % = 41 %. Until fire tests concerning shear capacity are carried out, Eq. (4) may be applied provided the first term is set equal to zero and the second is reduced to 55 %.

Fatique failure

Tests by Anderson and Anderson [7] indicate a high capacity even though 50 cycles of each load were applied. According to Chang and Kesler [12] the diagonal splitting strength for wet casted concrete after one million cycles of load is 45 – 65 % of the capacity wihtout cycles. It appears reasonable to assume that the anchorage capacity for hollow-core slabs as described in this paper, is at least 0,75 · 45 % = 34 %. Until fatique tests concerning shear capacity are carried out, Eq. (4)

may be applied provided the first term is set equal to zero and the second is reduced to 45 %.

QUALITY CONTROL CRITERIA

The free end slip is the best acceptance criterion concerning shear and anchorage. Following limits are proposed:

1. Moderate stresses:
   Max. 2,5 mm and highest average 2 mm
2. Very high stresses:
   Max. 2 mm and highest average 1,5 mm.

The slip is also an indication of the level of the concrete quality. It is not necessary to test the concrete compressive strength, because this strength is noe decisive for the shear capacity. Important is good consolidation of concrete around the strands.

In addition the surface of the concrete near the end of the slab, must be inspected. Around the strands there should be no distinct cracks caused by the production method. But cracks limited to the surface of the concrete do not reduce the capacity to any important degree.

CONLUSIONS

The scattering of the ultimate shear capacity is considerable, probably because of the production method of the slabs that may give very variable compaction around strands. Nevertheless, the number of tests is believed to be sufficient to give safe recommendations. Some of the capacities were low, because the elements were of second class quality.

The capacity is not the same for all types of hollow-core slabs which are reported in this paper. But, it is difficult to distinguish between the shear capacities of the different types, and it is unnecessary for practical design purposes.

Before failure, the element carries the load as a combination of beam- and arch action. Diagonal tension and/or shear bending introduce the failure. However, slabs with a short support at the end of the element collapse because of anchorage failure. The shear capacity consists of a concrete and reinforcement contribution. The latter is normally the largest.

These tests indicate that the examined hollow-core slabs have a very high shear capacity. Based on literature studies and the determined shear capacity formula, the fire resistance seems to be considerable for a period of 90 mintues. But the vertical end of the element must be protected against high temperature.

ACKNOWLEDGEMENT

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SAMMENDRAG

Artiklen gjengir resultatet av et stort antall prøvebelastninger av fabrikk-produserte hulldekkelementer i forspent ekstrudert betong. Undersøkelsen er ledet av Norges byggforskningsinstitutt og utført i samarbeid med Norges Betongvarefabrikkers Forbund og 7 elementfabrikker. Med basis i forsøkene er det utledet dimensjonerende skjærkraftkapasitet for linjelast og jevnt fordelt last. Brannmotstanden og utmattingsstyrken er vurdert. De observerte påkjenninger og en sannsynlig statisk modell er beskrevet. I tillegg er det foreslått en kvalitetskontroll.

Forsøkene har vist at elementene har en meget høy skjærkraftkapasitet. Basert på litteraturstudier og den utviklede kapasitetsformel for skjær, ser det ut som elementene har en betraktelig bæreevne inntil 90 minutters brannbelastning. Men den vertikale enden av elementet må være beskyttet mot høye temperaturer.

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