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Contents

FIRE RESISTANCE OF PRESTRESSED CONCRETE BEAMS

J. Boon
Th. Monnier
(IBBC)

Preface

The research on the fire resistance of prestressed concrete beams described in this report, was carried out by the Institute TNO for Building Materials and Building Structures (IBBC-TNO) with the financial support of the Netherlands Committee for Concrete Research (CUR) and partially also by the Netherlands Society of Concrete Fabricators (BFBN). The investigations were carried out in close cooperation with CUR Committee C 4. The research started in 1953. From empirical results and theoretical considerations a method was developed for predicting the fire resistance of a prestressed concrete beam. This was published (in Dutch) in CUR report 13 (1963). It appeared however that some 10 à 15% of the beams behaved appreciably worse than this prediction. Therefore the investigations were combined in order to get a better indication of the reason for this discrepancy. This part of the investigations relates mainly to I-shaped beams, and forms the main body of the present report.

Also in this report results are given of fire tests on T-shaped beams. These tests were carried out with financial support of BFBN.

In the end phase of the investigations Committee C 4 was constituted as follows:

H. van Tongeren, Chairman
J. Boon, Secretary
F. J. B. Barends
J. G. Hageman
J. Saveur
A. P. van Vugt
C. J. A. Warmenhoven
M. Kroes
G. Huyghe
J. G. Hageman, Mentor

In earlier phases P. J. van Tussenbroek and F. J. B. Barends held the office of president. Also G. A. de Boer, C. W. van Hoogstraten and C. van Boven participated in the work of the committee for some time.

The research was carried out by J. Boon of IBBC-TNO who also acted as secretary. Because of illness of the secretary the final report was written by Th. Monnier of the same institute.

FIRE RESISTANCE OF PRESTRESSED CONCRETE BEAMS

Summary

This report reviews research that has been carried out on prestressed concrete beams. This research is in fact the continuation of the investigations described in CUR report 13 "Fire tests on prestressed concrete beams". In this report a method was developed for calculating the length of time for which a prestressed concrete beam can be exposed to fire before failure occurs in consequence of the critical temperature of the prestressing steel being attained. In the fire tests performed for those investigations it appeared, however, that a number of beams did not conform to the rule and, instead, failed prematurely and suddenly. Those beams were called "rogues".

The present report deals with the cases where premature failure occurred in fire tests. These can be subdivided into two types, namely, the said rogues – which are encountered chiefly in I-section beams, in which continuous horizontal cracks develop in the web – and cases where slip of the pre-tensioned $\frac{1}{2}''$ strand tendons initiates premature failure. All the cases of premature failure exhibit the character of shear failure. On the other hand, the attainment of the critical temperature of the prestressing steel shows similarity with bending moment failure.

The research relating to the rogues is described in Chapter 2. This phenomenon is found to occur when, in consequence of the temperature differences that arise in a beam during a fire, "horizontal" cracks develop, along which there is deficient shear transmission capacity. Rogues are obviated if stirrup reinforcement is provided which can resist the whole shear force occurring during the fire. The procedure for designing the required stirrup reinforcement is given in Chapter 4. This calculation is, however, based on the assumption that no shear stresses at all can be transmitted across the "horizontal" crack. In reality it will therefore be possible to manage with less shear reinforcement. No properly justified opinion on the magnitude of the reduction can be given.

The research relating to cases of tendon slip was carried out mainly on T-section members with $\frac{1}{2}''$ pre-tensioned strands. These matters are discussed in Chapter 3. The beams in question were, for the most part, exposed to the fire over their entire length, as contrasted with the cases discussed earlier, where the ends of the beams remained cold. For the T-section members dealt with in Chapter 3 the occurrence of premature failure can be obviated by the following precautions:

a. Preventing the slip of the tendons, which can be achieved by:
   - anchoring the tendons one by one;
   - keeping the ends of the beams cold, or insulating them, for a distance of 1 meter, if $\frac{1}{2}''$ strands are used.

b. Installing stirrup reinforcement which is able to resist half the shear force that occurs during the fire. This reinforcement should extend into the compressive zone.

The design procedure for these stirrups is given in Chapter 4. The resistance between the slipping pre-tensioned strands and the concrete is very critical and may still become the deciding factor for the loadbearing capacity during the fire.

c. Improving the bond between the tendons and the concrete. Two possibilities were investigated:
   - the use of smaller-diameter strands, namely, $\frac{3}{8}''$; the slip is reduced in consequence;
   - disposing $\frac{1}{2}''$ strands in a concentrated arrangement, surrounded by one helical reinforcement; in this case the amount of slip is about the same as with $\frac{3}{8}''$ strands.

The fire resistance of the beams investigated in this research ranged from 75 to 90 minutes, except for the helically reinforced beams discussed in section 3.5, which were found to have about 110 minutes fire resistance.

The calculation of the fire resistance in the case where the critical steel temperature is attained (bending moment failure) is likewise given in Chapter 4.

All the fire resistance calculations presented in this report are based on:
- the requisite fire resistance;
- the standard fire temperature curve (Fig. 1);
- the load that is present during the fire;
- the material properties referred to the high temperature on reaching the fire resistance (see Figs. 24 and 25).

The main problem, which remains more or less unsolved, is that of the bond strength of steel and concrete under fire conditions. It has been established, however, that the use of $\frac{3}{8}''$ strands, or a helix of reinforcing steel around $\frac{1}{2}''$ strands in a concentrated arrangement, are good solutions for postponing the occurrence of failure in consequence of deficient bond strength.

Fire resistance of prestressed concrete beams

Chapter 1

INTRODUCTION

1.1 General

Research on the fire resistance of prestressed concrete beams (made of normalweight concrete) has been the subject of an earlier report, namely, CUR Report 13: "Fire tests on prestressed concrete beams". In that publication a method was developed with which it can be calculated for how great a length of time a prestressed concrete beam can be exposed to fire before it fails. In the fire tests performed for that research, however, it was found that a number of beams did not conform to the rule and, instead, collapsed prematurely and suddenly. For this reason those beams had to be rated as "mavericks" or "rogues", and for convenience the latter designation will be used here.

The object of the research reported here was to find an explanation for the premature failure and, if possible, to arrive at a method of calculation which, as regards prediction or verification of fire resistance, would also comprise these cases.

The rogues encountered mainly in tests on I-section beams are discussed in Chapter 2. Chapter 3 deals with the premature failure of so-called T-elements – prestressed concrete T-section members – which, in contrast with the rogue beams, were prestressed with pretensioned strand tendons. Finally, Chapter 4 presents the fire resistance calculation method for prestressed concrete beams, including the abovementioned cases of premature failure under fire conditions.

1.2 Fire tests

In the fire test a beam is placed in a test furnace. The temperature in the furnace is then raised in accordance with the so-called standard fire curve (time-temperature curve) (Fig. 1). This curve aims at providing the best possible approximation to the average temperature pattern in actual fires. It has been internationally recognized by the I.S.O. and has been adopted also as the basis in the Netherlands Standard NEN 1076 "Fire resistance of structures". The test beams are subjected to the fire tests while under their total characteristic loading; the temperatures in the concrete and in the steel are usually measured at various points with the aid of thermocouples. In addition, the deflections are usually also measured.

The principal object of the test, however, is to determine the *fire resistance*, i.e., the period of time during which the beam, while subjected to the total characteristic loading, can survive the fire test.

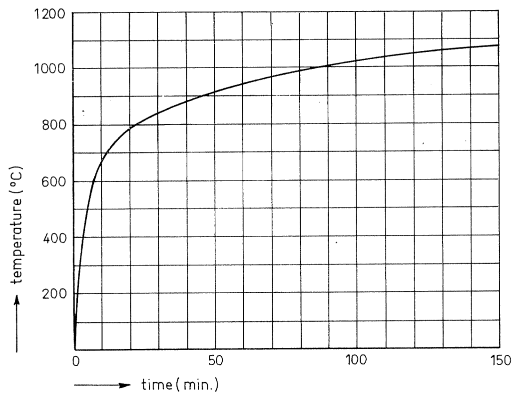


Fig. 1. Standard time-temperature curve.

1.3 Statement of problem; scope of research

The character of the failure or collapse of a beam in a fire test generally shows good agreement with that of a beam which, at normal temperature, is subjected to an increasing load until the failure bending moment is reached.

In the fire test, too, "yielding" of the reinforcement at a certain instant causes the deformations to increase very rapidly (Fig. 2). The concrete finally fails by crushing of the compressive zone. Before that happens, fragments of concrete usually become dislodged from the beam (spalling) and cracks develop. Horizontal or inclined cracks tend to occur more particularly at the level of the reinforcement or, in I-section beams, at the junction of the flange and web.

In general, the vertical cracks are formed later; just before collapse, wide vertical cracks appear (yielding of the steel), marking the occurrence of failure and the end of the period that determines the fire resistance. From that instant onward the steel, here more particularly the prestressing steel, is unable to develop the tensile force necessary for maintaining structural equilibrium.

The *critical temperature* is said then to have been attained in the steel. In CUR Report 13 it has been explained how in such cases the fire resistance can be determined in advance, while it also emerges that for this mode of failure it is possible to predict

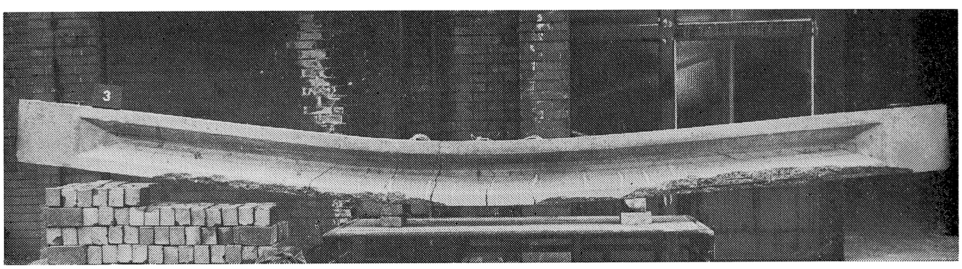


Fig. 2. Beam collapsed during a fire test as a result of the critical steel temperature being attained.

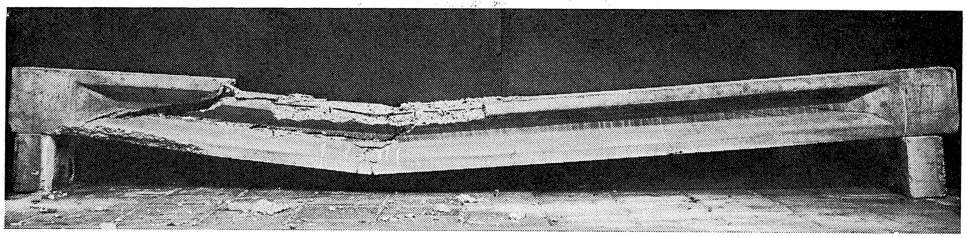


Fig. 3. "Rogue" collapsed during a fire test.

the fire resistance with reasonable accuracy. For the sake of completeness this calculation procedure will be presented in concise form in Chapter 4.

In the case of I-section beams the fire test sometimes runs a different course. It may happen that the test beam collapses suddenly after only a relatively short time (i.e., before any appreciable amount of deformation has occurred) and indeed long before the critical temperature can have been reached in the steel. It is such beams that are here referred to as "rogues" (Fig. 3).

The present report ties up with CUR Report 13 in so far as its treatment of the rogues is concerned. The supplementary research that was carried out for this purpose comprised, in the main, fire tests on prestressed I-section beams made of ordinary (normalweight) concrete and containing pretensioned wires. The object of the tests was more particularly to obtain a better insight into the cause of premature and sudden failure. This research led to the conclusions that after temperature stresses had caused small "horizontal" cracks to form in the web the shear stresses occurring there could no longer be effectively transmitted. It was found that this premature collapse can be prevented by providing the beam with stirrups. In order to find out how much stirrup reinforcement is needed, some fire tests were also performed on beams in which this reinforcement was varied. Finally, within this research context, a number of beams were loaded to failure at normal temperature. All these tests were performed on model beams, namely, 1:1.5 scale models of the fire test beams with which CUR Report 13 was concerned.

All the fire tests described in that report relate to I-section prestressed concrete beams with the following conditions of support. At one end of the fire test furnace the end of each beam protruded from the furnace and rested on a roller bearing. These beams therefore remained cold along a length of about 400 mm; while the other end of each of them was provided with thermal insulation for a distance of about 200 mm. Fig. 4 shows the furnace and gives further information on the arrangements at the two ends of the beams. A number of them were prestressed with post-tensioned tendons, the others were provided with pretensioned wires.

The I-section test beams used in the present supplementary research were in some cases provided with anchorages at the ends of the prestressing wires, although these were pretensioned. The beams in these tests were placed completely within the furnace, without any special insulation.

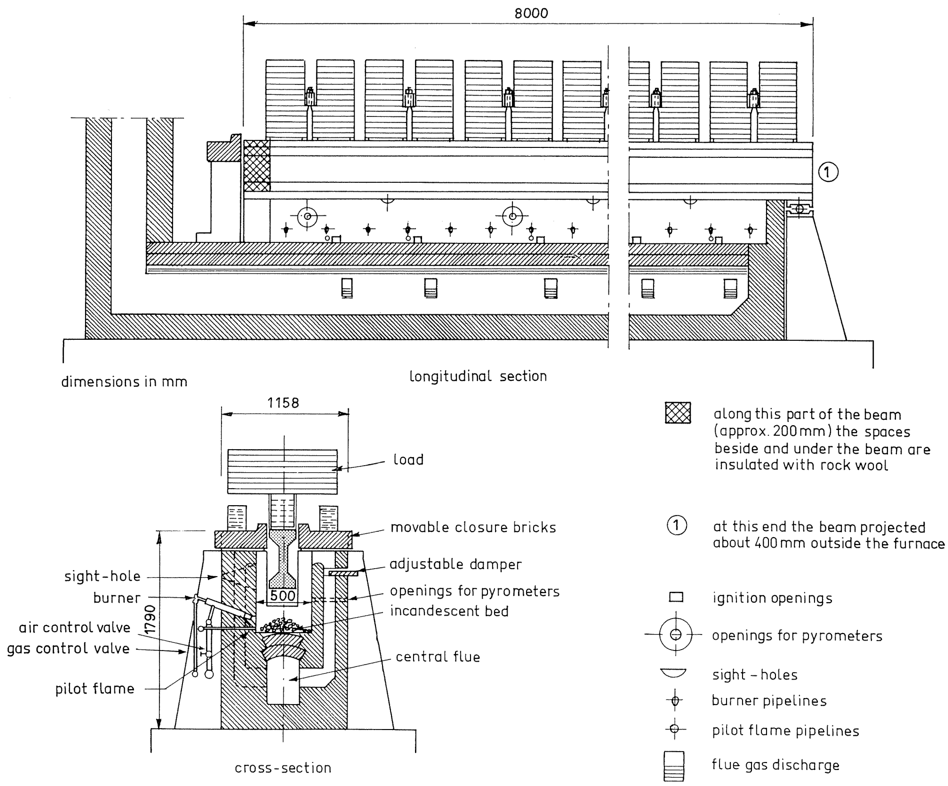


Fig. 4. Longitudinal and cross-section of the test furnace.

Fig. 5. Slip of 7-wire 12.5 mm strand in a T-beam exposed to fire along its entire length.

The T-elements, prestressed with pretensioned 7-wire strands of 12.5 mm diameter, were likewise placed completely within the furnace. In general, the two ends of each specimen were insulated for a distance of only 50 mm, so that these were more highly heated than the ends of the full-size I-section specimens envisaged in CUR Report 13.

There was found to be an appreciable decrease in fire resistance in those elements which contained either no stirrup reinforcement or only a light stirrup reinforcement, the most notable phenomenon in these cases being the slip of the pretensioned strands (Fig. 5). As a result, failure occurred mainly in the vicinity of the ends. This mode of failure can be prevented in various ways, namely:

- by insulating the ends of the beam;
- by installing sufficient stirrup reinforcement;
- by anchoring the strands;
- by adopting a more concentrated arrangement of the strands and enclosing them within a helix;
- by using strands of 9.6 mm instead of 12.5 mm diameter.

CHAPTER 2

ROGUES

2.1 General

The results of fire tests on about forty prestressed concrete beams have been described in CUR Report 13. These results, together with other experience obtained both in experimental research and in actual fires, indicate that 10 to 15% of the beams were "rogues" and that these occur mainly among I-section beams.

With regard to the set-up adopted for the further investigation of this phenomenon it was of course assumed that certain features of these specimens might be the cause of the occurrence of such rogue beams. It was endeavoured to arrange the tests in such a way that now this 10 to 15% proportion of rogues would not occur, but that by *exaggeration* of the feature in question each beam could in principle become a rogue.

2.2 Investigation and results

The tests were performed on I-section model beams with dimensions (except the length) reduced by a factor of 1.5 in relation to the full-size beams with which CUR Report 13 was concerned (Fig. 6). The material properties of the present test beams were as much as possible the same as those of the full-size prototype beams.

The fire tests on these beams were carried out in the so-called floor testing furnace of IBBC-TNO, the conditions for the model test beams being as much as possible equivalent to those originally adopted for the full-size beams (with regard to the arrangements at the ends of the beams see also Section 1.3). The loading to be applied to the beams undergoing the tests of course had to be adjusted in accordance with the rules of model similitude to the reduced scale of these test beams.* In the full-size

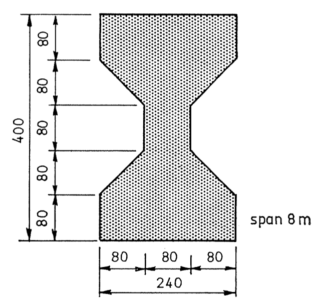


Fig. 6. Cross-section of full-size beams (CUR Report 13) span 8 m. (dimensions in mm).

* See CUR Report 21: "Measurements on the construction site".

beams the maximum bending moment was 90 kNm and the shear force was 45 kN. In the 1:1.5 scale model beams these values had to be:

$$M = 90/1.5^3 = 26.60 \text{ kNm}$$

$$T = 45/1.5^2 = 20 \text{ kN}$$

Each model beam was loaded with two 20 kN point loads applied at a distance of 1.33 m from the adjacent bearing. The loading arrangement is indicated in Fig. 7; the dead weight was neglected in each case.

In order to obtain the same temperature distribution in the model beams as in the full-size ones, the rate of heating had to be increased by a factor of $1.5^2 = 2.25$. For the same fire resistance of the two types of beam the time that elapses before collapse of the full-size beams must accordingly be 2.25 as long as for the model beams. Thus the fire resistance period up to the attainment of the critical steel temperature was 75 minutes for the full-size and 33.3 minutes for the model beams (see also Section 4.3.1).

It was, to begin with, assumed that the reason why some beams behaved as rogues must be sought in the web being too thin (web thickness 80 mm in the full-size and 53.3 mm scaled down in the model beams). Research was therefore first directed at this aspect: model beams with webs 30, 40, 50 and 70 mm thick were made (Fig. 8). The results of the fire tests on these beams are given in Table 1.

The model beams 1 to 8 had relatively very thin webs (30 mm) and attained a fire resistance of 37 minutes on average, corresponding to about 84 minutes in a full-size

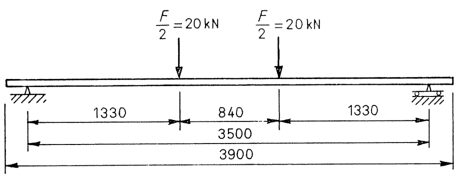


Fig. 7. Loading arrangement on model beam. (dimensions in mm).

beam. In these fire tests, therefore, the critical steel temperature was reached in all the specimens – there were no rogues. The fire resistance times found in these tests are in good agreement with those of the non-rogue beams in the earlier full-size tests. So a relatively thin web in an I-section beam is evidently not in itself the cause of rogue behaviour.

Another supposed possible causal factor of such behaviour was the premature and sudden spalling of the concrete cover. In order to induce such spalling and loss of cover to the steel, vertical holes of 10 mm diameter were locally drilled in the bottom flange of model beams 9 and 10 (Fig. 9). The object was suddenly to remove this part of the concrete cover by knocking it off with a steel bar 10 minutes after the start of the fire test.

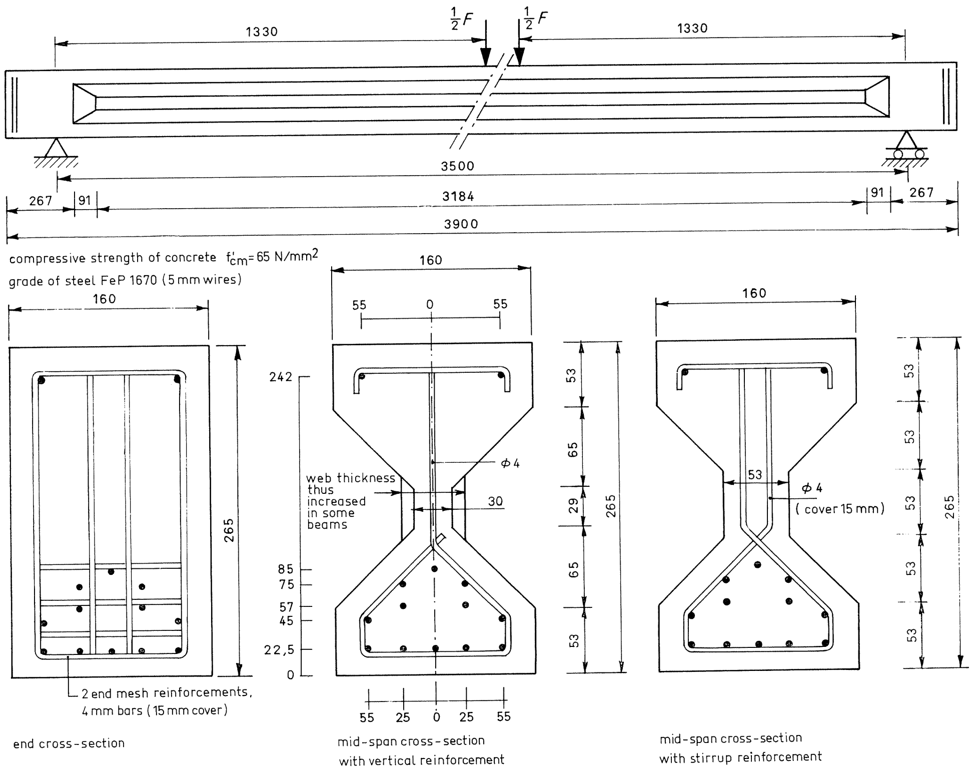


Fig. 8. I-section model beams.
(dimensions in mm).

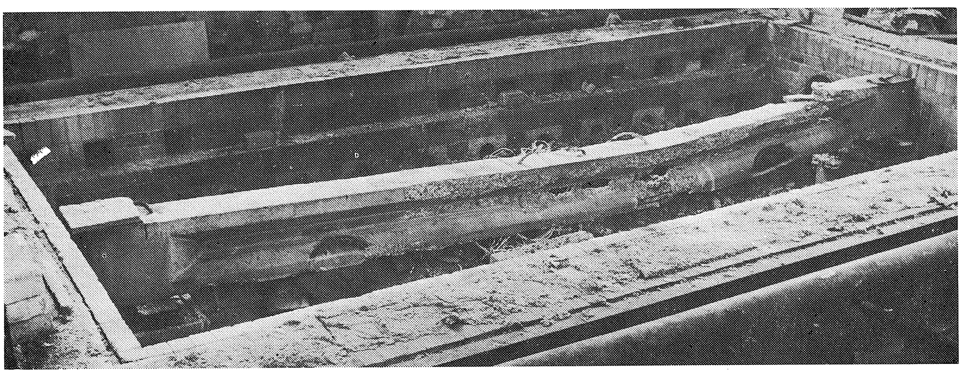


Fig. 9. Model beam 10, with vertical holes drilled in bottom flange at approximately quarter-span point, failed suddenly in a fire test.

The fire resistance of model beam 9 was indeed considerably diminished as a result of thus removing the concrete cover to the prestressing steel. The beam failed in the normal way, however. Understandably, the critical steel temperature was reached earlier because of the absence of the cover.

Table 1. Results of the fire tests on the first series of model beams with vertical reinforcement of 4 mm bars at 200 mm centres

model beam	web thick-ness (mm)	fire resis-tance (min)	particulars	comparison with full-size beams in CUR Report 13	conclusion
1	30	46	after 9 minutes the first spalling of concrete fragments occurs on beams 1 to 8	– full-size beams had fire resistance of 79 and 76 minutes respectively (A5 and B5), i.e., 35 and 33 minutes in models	– fire resistance agrees with full-size-beams
2	30	37	types of failure:	– calculated fire resistance for full-size beams 75 minutes, i.e., 33.3 minutes for model beams	– critical steel temperature is attained
3	30	41	– beams 1, 2, 4 and 5: top flange crushed over a distance of 1 m		– no rogues
4	30	34	– beam 3 was supported on packings; there were "yield cracks"; compressive zone of concrete did not fail		
5	30	44	– beams 6, 7 and 8: compressive zone of concrete failed; slip of the wires occurred in final stage		
6	30	$29\frac{1}{2}$			
7	30	33			
8	30	35			
9	30	$14\frac{1}{2}$	– drilled holes – cover knocked off; local failure occurred		no rogue; prestressing steel directly exposed to fire
10	40	10	– drilled holes – attempted to knock off the cover: unsuccessful – beam failed suddenly at other end		rogue
11	40	5	– saw-cuts in web; bottom flange displaced 20–30 mm – beams 11 and 12 loaded with dead weight – beams 13 and 14 loaded with jacks		all four beams with a horizontal crack were rogues
12	50	7			
13	70	9			
14	70	$10\frac{1}{2}$			

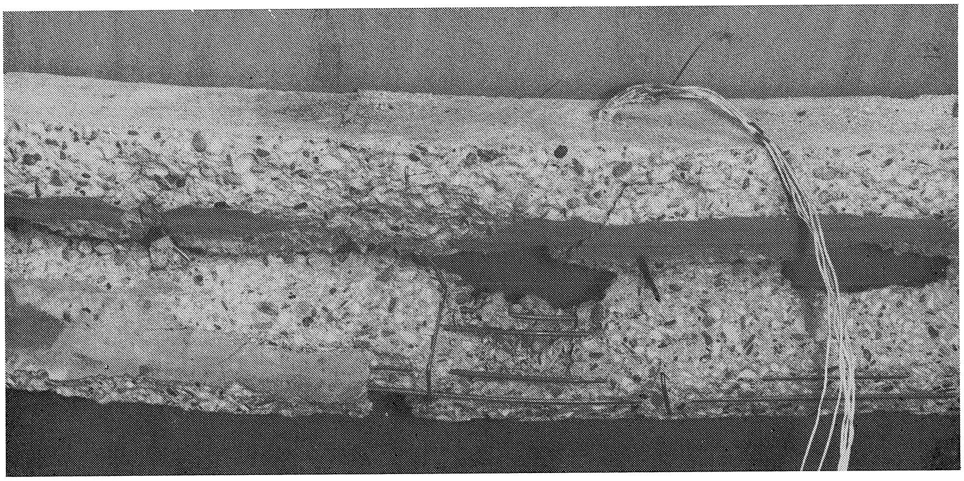


Fig. 10. Displacement of top flange in relation to bottom flange in model beam 10.

Model beam 10 collapsed suddenly at the first knock (which, incidentally, failed to dislodge the cover); in this case failure developed at the other end from that where the beam was knocked (Fig. 9). A phenomenon similar to that which characterized the rogues in the earlier tests was thus manifested here.

This last-mentioned test strengthened the already existing impression that the failure associated with the sudden collapse of the beams has the character of shear failure, as contrasted with the failure behaviour due to the critical steel temperature being reached, which has the character of bending moment failure. After the test the top flange of the beam which had suddenly collapsed was found to have undergone a substantial displacement (of several centimetres) in relation to the bottom flange (Fig. 10). In addition, a series of small oblique cracks of the type shown in Fig. 11 had occurred in the web already in an early stage of the test. The erratic character of these cracks determines to what extent the web of the beam is still capable of transmitting shear stresses.

The development of the "horizontal" crack referred to above – which actually consists of a series of small oblique cracks of which the combined effect is comparable to that of a horizontal crack – can be explained as follows.

At first only the temperature of the outer layer of the concrete rises considerably, this rise being more pronounced at the underside of the beam than at the top. As a result of the great difference in temperature between the as yet almost cold core and the outer layer, internal stresses of considerable magnitude are produced, both in the horizontal and in the vertical direction. With regard to the formation of "hori-

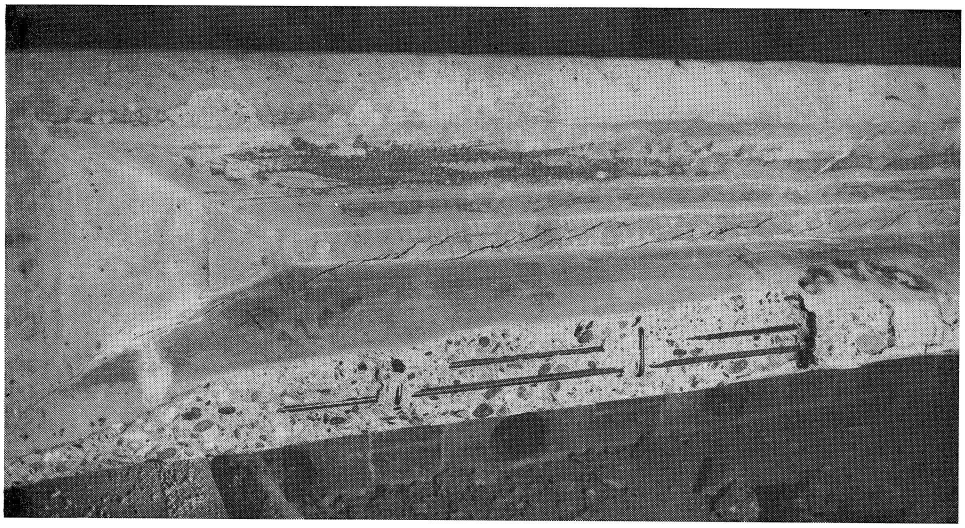


Fig. 11. Typical cracking due to temperature gradient in the web of a model beam.

zontal" cracks the stresses acting in the vertical direction will be considered in more detail (Fig. 12).

From the equilibrium of the horizontal section it appears that – on the assumption that plane sections remain plane – compressive stresses develop in the vertical direction in the outer layer of concrete, while tensile stresses develop in the cold core of the beam. These stresses are of such magnitude that small cracks will develop, in the horizontal direction, in the interior. As a result of this cracking, the stresses

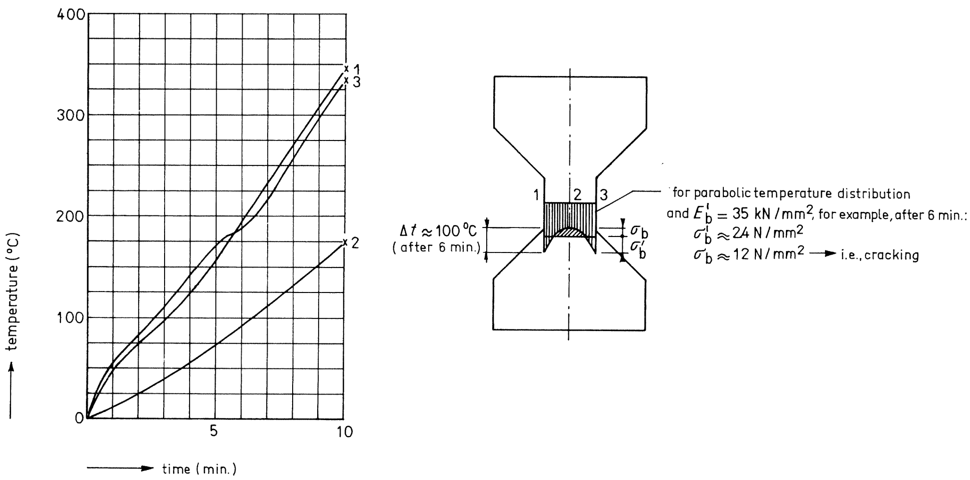


Fig. 12. Example of temperature distribution in the web of a model beam during the fire test.

in the outer layer will of course largely disappear. It stands to reason that, in consequence of notch effect, the internal cracks will spread to the external surface of the concrete. As already stated, the question is to what extent these erratic cracks allow the concrete to transmit shear stresses.

To find the answer to this question, fire tests were performed on four model beams (11 to 14) which had been left over from the preceding series and in which an "exaggerated" smooth crack was deliberately formed in the middle of the web. For that purpose the web (including the vertical reinforcement) was sawn through horizontally

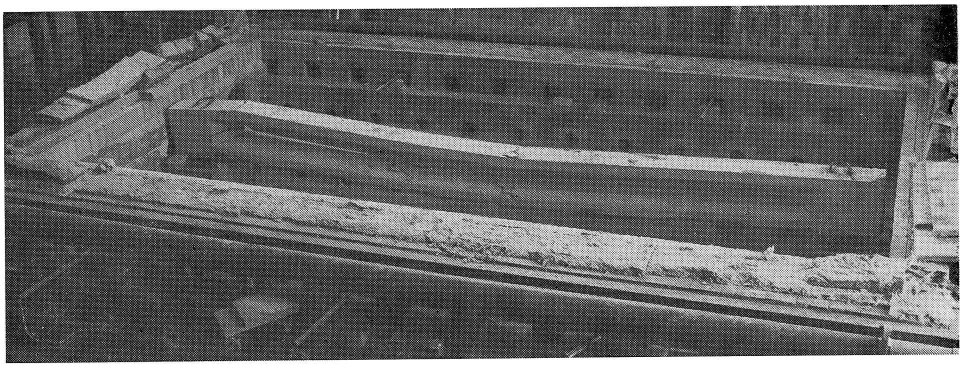


Fig. 13. Model beam with horizontal saw cut in the web, here seen after the fire test.

between the end blocks of the beam. The saw cut was, however, interrupted for a distance of 0.5 m at the points of load application. In these beams the vertical reinforcement was therefore no longer intact.

The loading which acted during the fire test was the same as that applied in the tests on the model beams already discussed (Fig. 7). In the case of the model beams 11 and 12 the loading consisted, as had been the normal procedure in the preceding fire tests, of dead weight. The loading on model beams 13 and 14 was applied by means of jacks. This was done because, so far as known, jacks are always used in similar tests performed in other countries and because this problem of rogue beams does not arise there.

In the course of the fire tests the four model beams 11 to 14 failed as rogues: failure occurred after periods ranging from 5 to 10.5 minutes, corresponding to approximately 11–24 minutes for a full-size beam (Fig. 13). After failure it was found that, in the region where failure had developed, the top flange had undergone 20–30 mm displacement in relation to the bottom flange. In all cases it was established after the test that slip of the prestressing steel had occurred, though no slip was observed during the test itself. From these tests it emerges that a continuous horizontal crack in the web of an I-section beam may give rise to rogue behaviour. The end blocks evidently cannot adequately compensate for the absence of the interconnection of the top and bottom parts of the beam in the I-section part thereof. Load application through jacks instead of by dead weight was found to be of no effect with regard to this.

It therefore appeared justified to attribute rogue behaviour with a very high degree of probability to the above-mentioned phenomenon. An obvious assumption was that the problem of rogues could be prevented by installing (more) stirrup reinforce-

Table 2. Results of fire tests performed on model beams with an artificially formed horizontal crack for investigating the effect of vertical reinforcement against rogue behaviour

model beams (web thickness 53 mm)					full-size beams (web thickness 80 mm)		
model beam	reinforcement in web (mm)		(‰)	fire resistance* (min)	comparable reinforcement in web (mm)		fire resistance* deduced from model beams (min)
1A	none		–	8	–		18
1B	none		–	7½	–		17
2C	stirrups	∅4–333	1.42	15	stirrups	∅6–500	34
3B	stirrups	∅4–200	2.36	24½	stirrups	∅6–300	55
3C	stirrups	∅4–200	2.36	31**	stirrups	∅6–300	70**
5C	vertical reinforcement	∅4–100	2.36	16½	vertical reinforcement	∅6–150	37

* calculated fire resistance for full-size beams 75 minutes and for model beams 33.3 minutes

** furnace temperature substantially lower than required to conform to standard time-temperature curve

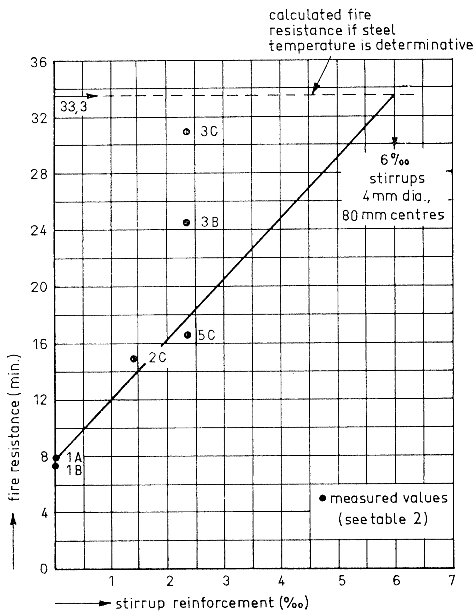


Fig. 14. Effect of the quantity of stirrup reinforcement on the fire resistance of model beams with an artificially produced horizontal crack.

ment in the web. For this reason another six model beams with different amounts of such reinforcement were tested. Table 2 gives information on the stirrup reinforcement provided (if any). The web thickness adopted for these six model beams was $80/1.5 = 53$ mm. The other dimensions of the beams, and the positioning of the prestressing wires in them, corresponded to those of the model beams discussed earlier on.

In the six model beams under consideration the horizontal crack that may develop in the web under fire conditions was preformed by embedding a 0.02 mm thick strip of copper at mid-depth of the web, and extending the whole length of the beam, in each beam at the time of concreting. In order to make sure that premature failure would on no account be caused by slip of the prestressing wires, these were provided with anchorages. Fire testing procedure and the loading applied during the test were the same as in the tests performed on the model beams already discussed.

The results obtained with this series of six model beams are given in Table 2 and in Fig. 14.

2.3 Effect of stirrup reinforcement

The beams without stirrups showed reasonably good agreement with the rogues among the full-size beams. Increasing the stirrup reinforcement or installing vertical reinforcement at the centre of the web did indeed result in better fire resistance. The number of tests carried out is rather limited, however, and for this reason the results must be approached with some caution. It is nevertheless evident that a relatively substantial amount of stirrup reinforcement is needed to enable the critical temperature to be attained in the prestressing steel. Linear extrapolation of the results presented in Fig. 14 leads to the conclusion that about 6‰ of vertical reinforcement, referred to the horizontal sectional area of the web, is necessary to enable the critical steel temperature to be reached (for the model beam this vertical reinforcement corresponds to 4 mm stirrups at 80 mm centres). From the calculation of the fire resistance (see Chapter 4) it follows that the critical steel temperature in the model beams occurs after 33.3 minutes. With reference to the results of these tests it must of course at once be conceded that the artificial smooth crack performed in these model beams is much more unfavourable as regards shear stress transmission than the erratic longitudinal cracks that develop in an actual fire. The test results under consideration are thus indeed on the unfavourable side in relation to what happens in reality; but it is not possible to say how much too unfavourable.

After the fire tests on model beams provided with an artificially preformed crack, as described in Section 2.2, there were still five of these model beams left for possible testing. As the results obtained led to the opinion that fire tests on these beams would yield practically no further worth-while information on fire resistance, it was considered what other tests would be most meaningful for testing the "rogue" hypothesis. As known, the "horizontal crack" develops at a very early stage in the fire test. If it can be shown that, a as result of this crack, the failure load comes very close to the working load of service load of the beam, it can reasonable be presumed that the beam will collapse at an early stage in the fire test. Since the calculation of the safety against failure of such a beam with a longitudinal crack – resulting in the formation of an unreinforced "top horizontal member" – and with virtually unreinforced end blocks gives rather doubtful results, it was decided to determine the failure load of the remaining five model beams at normal temperature. The results of these tests are given in Table 3.

Table 3. Strength of model beams provided with an artificially formed horizontal crack and loaded at normal temperature

model beam	reinforcement in web (mm)		(‰)	failure load F (kN)
1C	none		–	52.5
2A	stirrups	∅4–333	1.42	65.3
4C	vertic. reinf.	∅4–167	1.42	67.7
3A	stirrups	∅4–200	2.36	68.0
5A	vertic. reinf.	∅4–100	2.36	73.5

The sequence of events in determining the failure load was practically the same in all the model (Fig. 15):

- for $F \approx 25$ kN a crack became visible at "a";
- for $F \approx 30$–35 kN cracks developed at "b";
- for $F \approx 35$–40 kN cracks developed at "c";
- at failure there was crushing of the concrete at the points "d"; the deflection was then about 100 mm and the displacement of "g" in relation to "h" was about 10 mm.

The higher failure load of model beam 4C as compared with 2A and the higher failure load of model beam 5A as compared with 3A is due to the fact that in the final stage

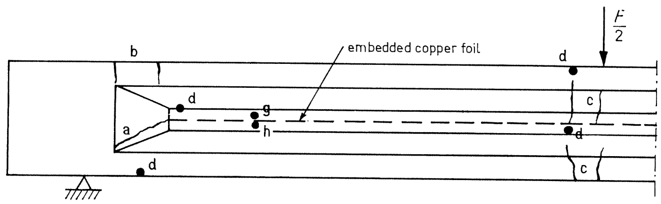


Fig. 15. Schematic representation of the phenomena observed in model beams, provided with an artificial horizontal crack, in the loading test at normal temperature.

the concrete cover spalled off the stirrups, whereas the cover to the vertical reinforcement disposed at the centre of the web remained intact. Fig. 16 shows the failure pattern developed in model beam 4C.

The failure load of the model beam without stirrups which is comparable with the rogues in the fire tests was 52.5 kN. The theoretical failure load of the model beam

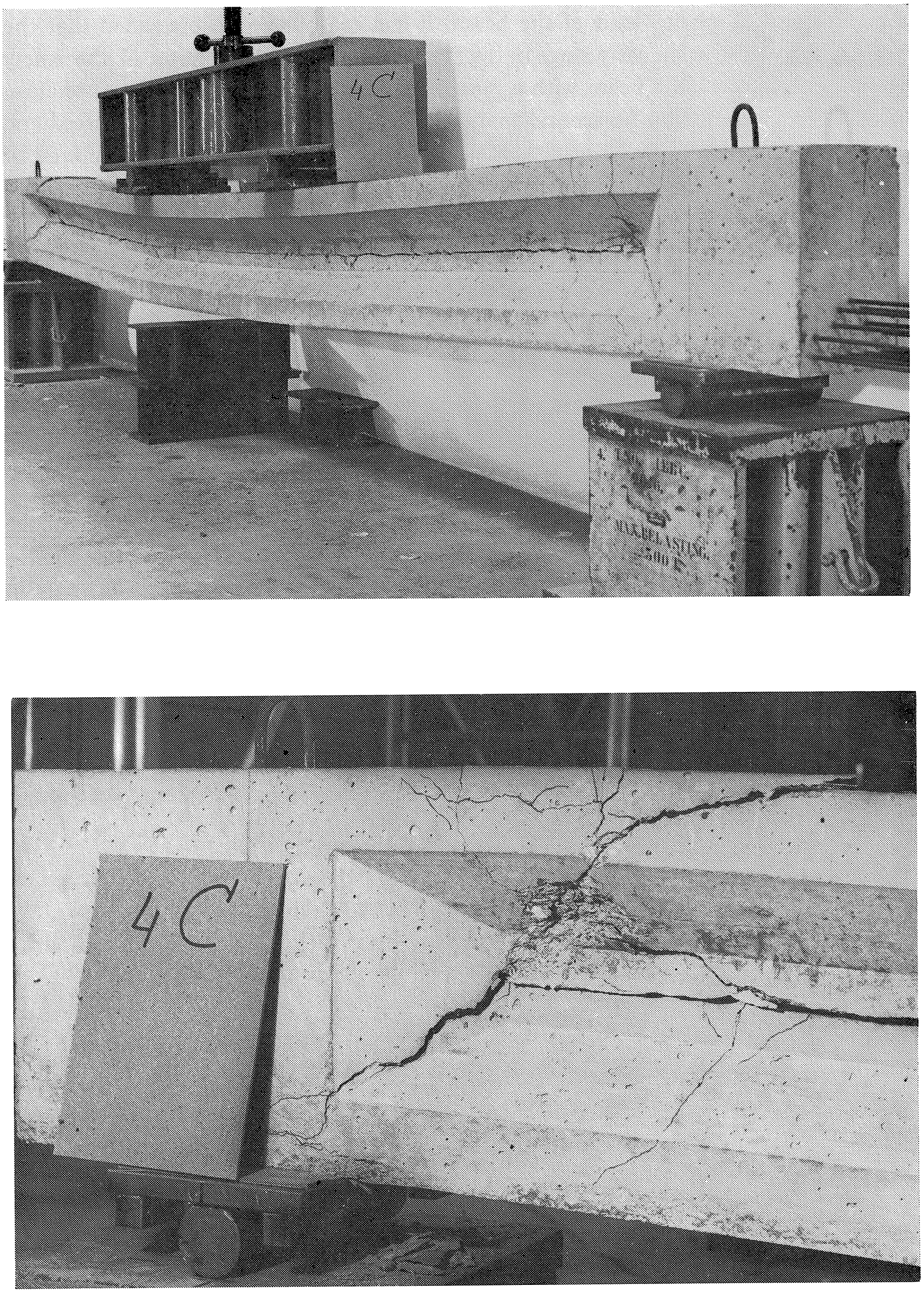


Fig. 16. Testing a model beam (beam 4C) with artificial horizontal crack to failure at normal temperature (see also Figs. 3 and 13).

without horizontal crack is approximately 110 kN, and the working load in the fire test was 40 kN. Hence it can be inferred that the failure load is greatly reduced in consequence of the horizontal crack and closely approaches the working load, so that the results obtained can indeed be regarded as supporting the accuracy of the explanation offered for the occurrence of rogues.

2.4 Interpretation of the fire tests

A number of elementary calculations were carried out with a view to interpreting the fire test results.

The object of designing a structure or a structural member with regard to fire resistance must of course be to ascertain whether a specified criterion can be fulfilled. Netherlands Standard NEN 1076 "Fire resistance of structures" defines what is precisely to be understood by fire resistance. The standard time-temperature curve (Fig. 1) has been adopted from that Standard, which moreover states that the required fire resistance must be attained with the total characteristic load (i.e., the sum of the dead weight, permanent load and complete live load) acting on the structure or member.

In the relevant calculations it is of course necessary to base oneself on the material properties that are applicable at the instant when the structure or member reaches its fire resistance.

With the aid of the lattice analogy (see Section 4.4.1) it is found that for a margin $\gamma = 1$ with regard to the shear force $T = 20$ kN (acting during the fire test) a quantity of stirrup reinforcement is required with a cross-sectional steel area equal to 7.6‰ of the horizontal sectional area of the web. Having regard to what has been said above, this is in good agreement with the 6‰ found by extrapolation of the measured values (Fig. 14).

In calculating the above-mentioned stirrup reinforcement the favourable effect due to the presence of the end blocks was not taken into consideration because loss of this favourable factor may occur in a fire. A reduction factor of 0.75 was applied to the yield stress at normal temperature ($= 300$ N/mm^2) for the stirrup steel. This reduction corresponds to a steel temperature of 450 °C occurring in the stirrups at the instant when the beam reaches its fire resistance and the critical temperature is attained in the prestressing steel (33.3 minutes fire resistance, see Fig. 24).

Furthermore, a completely smooth continuous horizontal crack was assumed to develop, as contrasted with the rather erratic crack pattern that actually arises in the fire test.

2.5 Conclusions

The conclusions with regard to the rogues are as follows:

a. Rogues occur when, in consequence of the temperature differences arising in the beams during a fire, "horizontal" cracks develop along which adequate transmission of shear forces is not possible.

b. The occurrence of rogues is avoided if a stirrup reinforcement is provided which is able to resist the entire shear force acting during the fire. In reality less stirrup reinforcement will suffice because the horizontal crack is not "smooth", but follows an erratic path. It is not possible to state how much this reinforcement can thus be reduced, as insufficient data are available for that.

Chapter 3

T-ELEMENTS

3.1 General

When prestressed "double T" (or "TT") precast concrete structural elements began to be extensively used in prefabricated construction, it had to be considered to what extent the experience obtained in fire tests performed mainly on I-section beams could be applicable to the TT-elements. Of course, it was also a matter of particular interest to know whether "rogues" were liable to occur with this cross-sectional shape as well. At the time it was not possible to predict whether the factors causing rogue behaviour would be discovered and, if so, how long it would taken to achieve this. It was therefore considered necessary to perform fire tests relating to TT-elements. These tests were carried out in collaboration with the Association of Precast Concrete Manufacturers in the Netherlands (BFBN).

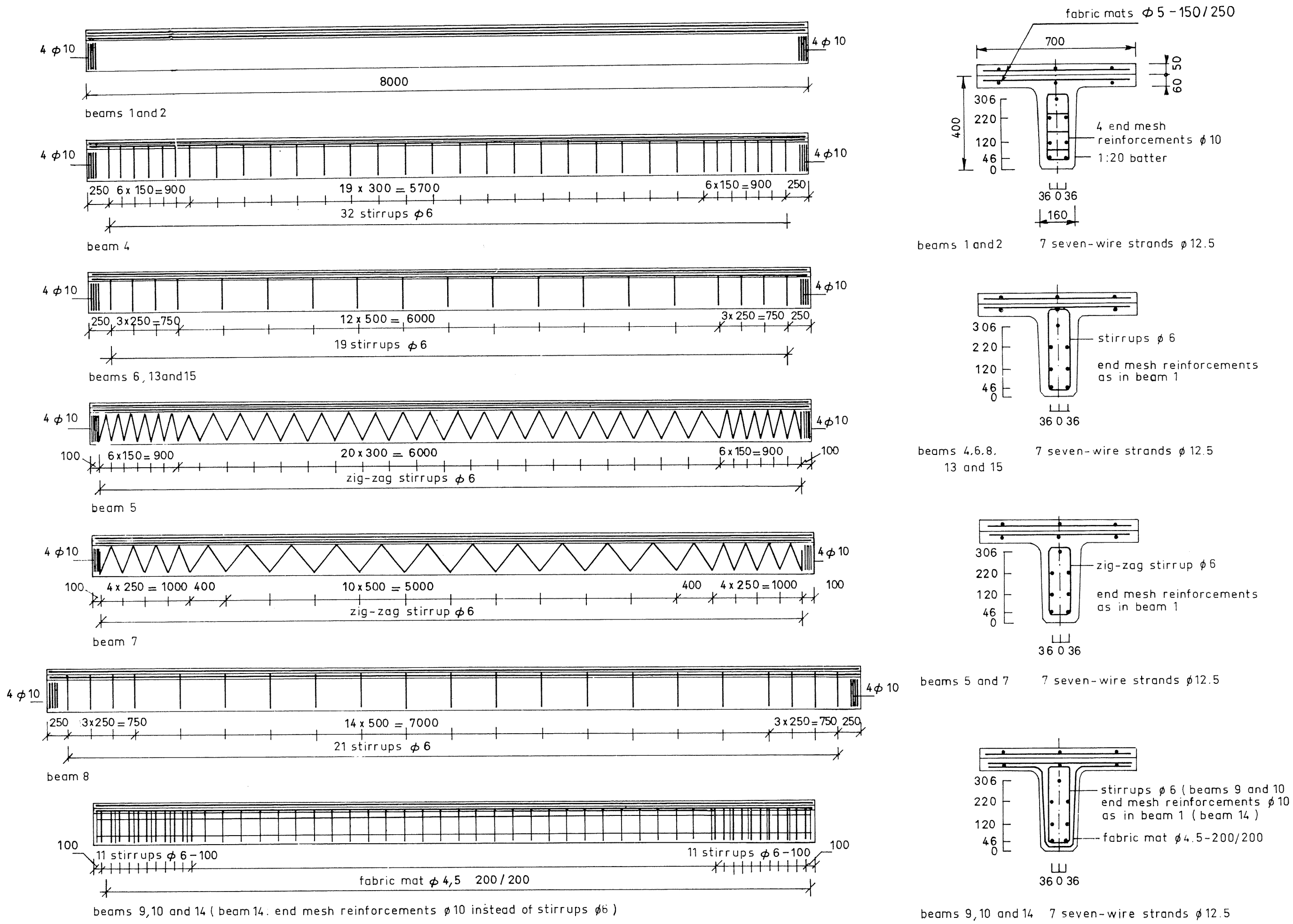
3.2 Research and results

The fire tests were performed on T-elements which could be expected to behave similarly to TT-elements.

The manner of fire-testing the T-elements calls for the following preliminary comment. In Holland and also (so far as the present authors are aware) in other countries it had hitherto been the standard practice to support the test beams on bearings located outside the furnace or to protect the ends of the beams with rock wool (see Fig. 4), so that the ends remain at low temperature. The reason for this practice is that the furnace wall, consisting of relatively soft refractory brick, is generally unable to serve as a bearing. This arrangement was evidently never regarded as other than acceptable, because the object of fire tests was considered primarily to consist in determining the critical temperature of the prestressing steel.

In the investigations on the T-elements, however, it was clear from the outset that the conditions occurring in actual practice would be better simulated by exposing the beams to the fire along the greatest possible length. Since the test furnace of the IBBC-TNO did allow the beams to be supported on the furnace wall, this testing procedure was accordingly adopted. The results of the test indicate that this procedure has some important consequences with regard to, among others, beams prestressed with 12.5 mm diameter pretensioned 7-wire strands.

The tests described in this chapter were concerned mainly with T-elements with a depth of 400 mm and a flange width of 700 mm (T 40/70). Because of the dimensions of these structural members the test furnace had to be widened (1.40 m internal dimension). Most of the T-elements were prestressed with 12.5 mm diameter 7-wire strands, while the remainder were prestressed with 9.6 mm diameter 7-wire strands


4 ϕ10
4 ϕ 10
8000
beams 1 and 2
250 6 x 150 = 900
19 x 300 = 5700
6 x 150 = 900 250
32 stirrups ϕ6
beam 4
250 3x250 = 750
12 x 500 = 6000
3x 250 = 750 250
19 stirrups ϕ6
beams 6, 13 and 15
100 6x150 = 900
20 x 300 = 6000
6 x 150 = 900 100
zig-zag stirrups ϕ6
beam 5
100 4 x 250 = 1000 400
10 x 500 = 5000
400 4 x 250 = 1000 100
zig-zag stirrup ϕ6
beam 7
250 3x250 = 750
14 x 500 = 7000
3 x 250 = 750 250
21 stirrups ϕ6
beam 8
100 11 stirrups ϕ 6 – 100
11 stirrups ϕ 6 – 100 100
fabric mat ϕ 4,5 200 / 200
beams 9, 10 and 14 (beam 14. end mesh reinforcements ϕ10 instead of stirrups ϕ6)
fabric mats ϕ 5 – 150 / 250
700
60 50
400
306
220
120
46
0
4 end mesh reinforcements ϕ 10
1:20 batter
36 0 36
160
beams 1 and 2 7 seven-wire strands ϕ 12.5
stirrups ϕ 6
end mesh reinforcements as in beam 1
beams 4, 6, 8, 13 and 15 7 seven-wire strands ϕ 12.5
zig-zag stirrup ϕ 6
end mesh reinforcements as in beam 1
beams 5 and 7 7 seven-wire strands ϕ 12.5
stirrups ϕ 6 (beams 9 and 10 end mesh reinforcements ϕ 10 as in beam 1 (beam 14)
fabric mat ϕ 4.5 – 200/200
beams 9, 10 and 14 7 seven-wire strands ϕ 12.5

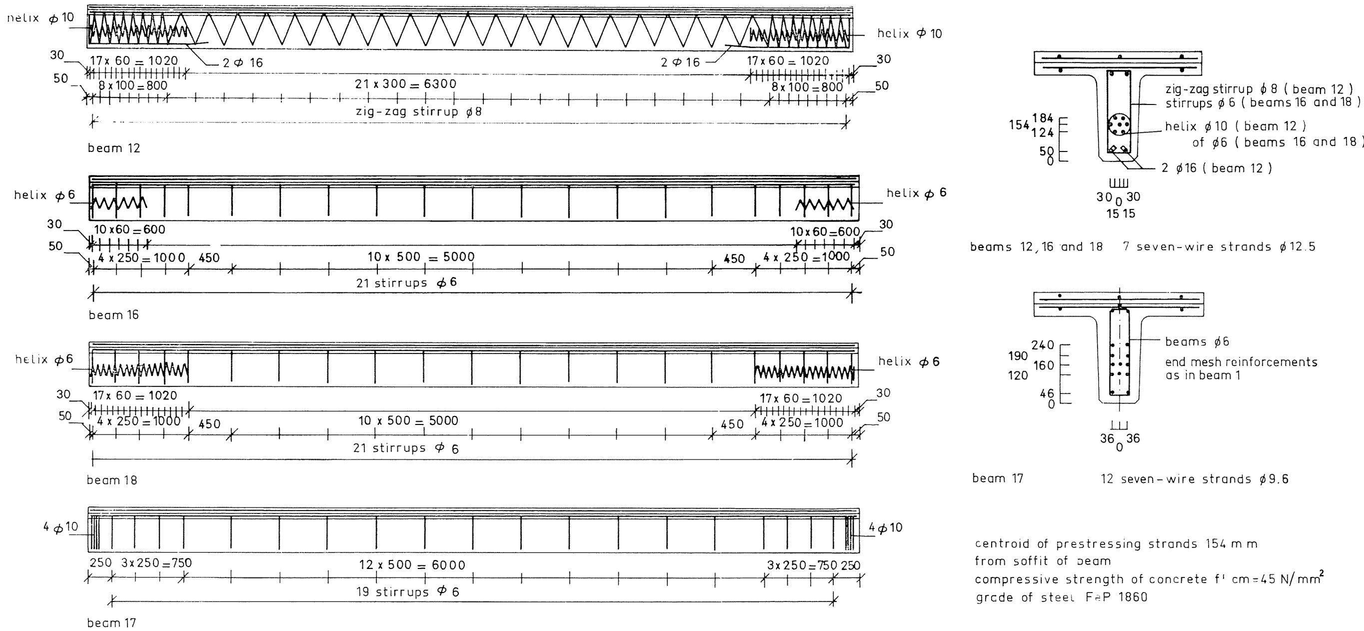


Fig. 17. Test beams T 40/70 for investigation of fire resistance of T-elements.

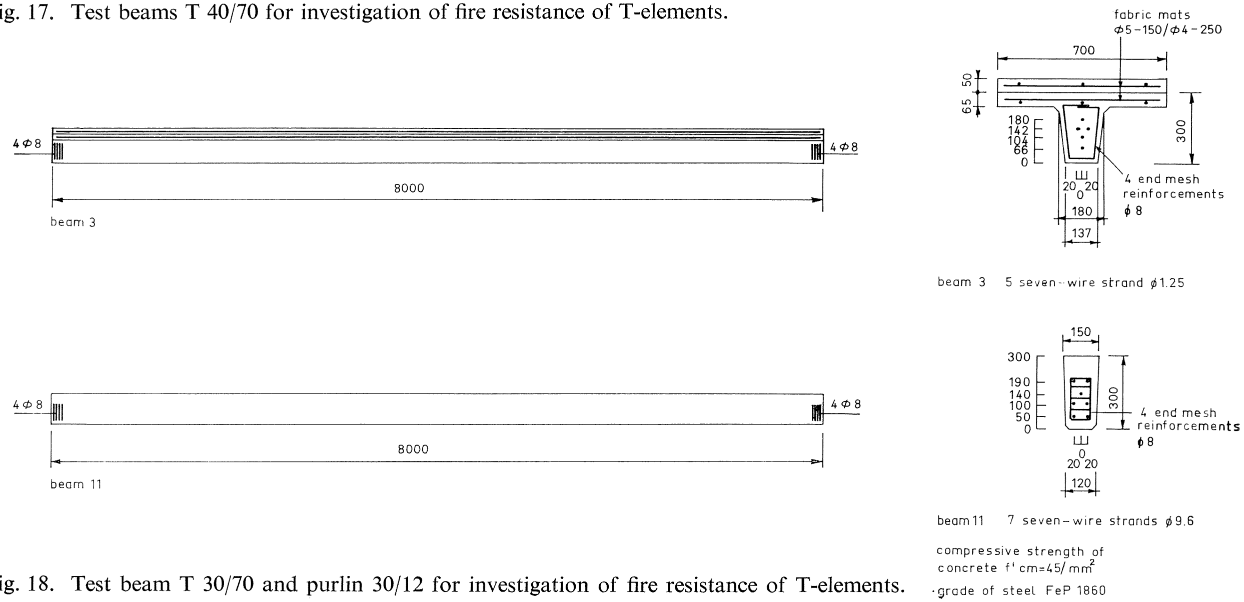


Fig. 18. Test beam T 30/70 and purlin 30/12 for investigation of fire resistance of T-elements.

Table 4. Data and results of fire tests on T-elements

test beam	prestressing steel (7-wire strands)	conventional reinforcement [1])	measured fire resistance (min)	variation in measured steel temperature (°C)	average slip of prestressing steel (mm/5 min)	maximum slip of prestressing steel at one end of strand (mm/5 min)	particulars [3])	age at testing (days)
1	7 strands ∅12.5	4 end mesh reinf. ∅10	44	345–175	2.0	3.5	140 mm at each end of beam protected	56
2	7 strands ∅12.5	4 end mesh reinf. ∅10	39	265–135	1.0	2.0	450 mm at one end of beam protected, slip and failure near this cold end	62
3	5 strands ∅12.5	4 end mesh reinf. ∅8	22	–	3.5	4.75	different cross-section CBR 30/70	67
4	7 strands ∅12.5	4 end mesh reinf. ∅10 stirrups ∅6-150; ∅6-300	66½	645–280	1.3	2.1		49
5	7 strands ∅12.5	4 end mesh reinf. ∅10 zig-zag st. ∅6, $s = 150$; 300 [4])	53½	430–225	1.5	2.5		61
6	7 strands ∅12.5	4 end mesh reinf. ∅10 stirrups ∅6-250; ∅6-500	60	495–210	1.3	1.7		64
7	7 strands ∅12.5	4 end mesh reinf. ∅10 zig-zag st. ∅6, $s = 250$; 500 [4])	74	605–300	–	–	strands anchored	70
8	7 strands ∅12.5	4 end mesh reinf. ∅10 stirrups ∅6-250; ∅6-500	82½	650–335	0	0	tested with cold ends 600 mm in length length of beam 9 m	61
9	7 strands ∅12.5	11 stirrups ∅6-100 fabric mesh ∅4.5-200	82	565–270	0.8	1.1		66
10	7 strands ∅12.5	11 stirrups ∅6-100 fabric mesh ∅4.5-200	80	645–335	–	–	strands anchored	70
11	7 strands ∅ 9.6	4 end mesh reinf. ∅8	50	440–305	0.6	0.8	purlin RNP 30/12	74
12	7 strands ∅12.5	helix ∅10 over 1.0 m [5]) zig-zag st. ∅8, $s = 100$; 300 [4])	102	469	0.6	0.7	concentrated prestress	–
13	7 strands ∅12.5	4 end mesh reinf. ∅10 stirrups ∅6-250; ∅6-500	53	418–230	3.0	3.5	no protection	97
14	7 strands ∅12.5	4 end mesh reinf. ∅10 fabric mesh ∅4.5-200	75½	578–269	1.0	1.3		109
15	7 strands ∅12.5	4 end mesn reinf. ∅10 stirrups ∅6-250; ∅6-500	56	470–240	1.1	1.3	artificially dried	133
16	7 strands ∅12.5	helix ∅6 over 0.6 m [5]) stirrups ∅6-250, ∅6-500	117	–	0.3	0.5	concentrated prestress	68
17	12 strands ∅ 9.6	4 end mesh reinf. ∅10 stirrups ∅6-250; ∅6-500	95	–	0.4	0.55		74
18	7 strands ∅12.5	helix ∅6 over 1.0 m [5]) stirrups ∅6-250; ∅6-500	109	–	0.8	1.0	concentrated prestress	78

[1]) dimensions in mm unless otherwise stated
[2]) relates to end of beam where larger average slip occurred
[3]) if not otherwise stated, each end of the beams was provided with protective insulation over a length of 50 mm against direct heating
[4]) zig-zag st. = zig-zag stirrup
[5]) all the helixes had a pitch of 60 mm and an outside diameter of 85 mm

(Table 4 and Figs. 17 and 18). With one exception (beam 3), these T-elements had been made by Nederlandse Spanbeton Maatschappij B.V., Alphen aan de Rijn. In all cases they were provided with a subsequently added 50 mm thick concrete topping co-operating structurally with the top (compression) flange.

The beams as used in actual practice, with 12.8 m effective span and 0.7 m centre-to-centre spacing, were designed for a superimposed floor load of 4 kN/m^2 (see Appendix A). The available test furnace could accommodate beams up to 8 m in length, and this was the length accordingly adopted for the test beams (except beam 8). By employing a more concentrated loading arrangement near mid-span it was ensured that the stresses occurring in the test beams due to bending moment and shear force were approximately equal to those in the actual full-length beams. The loading arrangement is shown in Fig. 19.

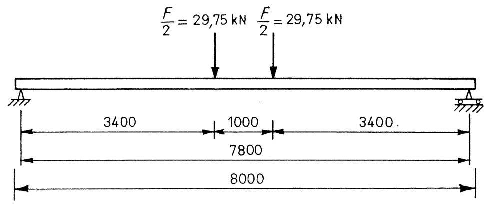


Fig. 19. Loading arrangement on the test beams T 40/70. (dimensions in mm).

The beams were simply-supported on a fixed and a roller bearing. As already stated, these bearings were installed on the end walls of the furnace, the effective span being 7.8 m. In order to expose the beams to direct heating along the greatest possible proportion of their length, their ends were protected with rock wool only over a length of 50 mm.

Although it was presupposed that in practice the beams would always contain reinforcement in the form of conventional stirrups, zig-zag stirrups or fabric – though in a quantity that was not or inadequately known – it was nevertheless decided to start with the extreme case, namely, beams containing no conventional reinforcement at all. The only non-tensioned steel in these specimens consisted of four small sets of mesh reinforcement (composed of 10 mm bars of steel grade FeB 220) to prevent splitting of the concrete (Figs. 17 and 18), but negligible as stirrup reinforcement. The results obtained with two of these beams were disappointing: the fire resistance was found to be 44 minutes and 39 minutes respectively, whereas on the basis of the attainment of the critical temperature in the prestressing steel a fire resistance of the order of 75 minutes could have been expected (see Section 4.3.2).

Test 3 related to a beam with different cross-sectional dimensions (T 30/70); its fire resistance was only 22 minutes (see Table 4 and Fig. 18). The load acting on the beam during the fire test had been calculated in the same way as for the T 40/70 specimens.

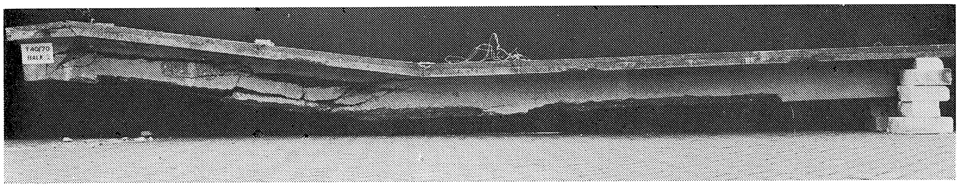


Fig. 20. Test beam 2 failed as a result of slip of prestressing strands.

In the premature failure of the above-mentioned three beams there was sudden shear-like failure beside one of the bearings (Fig. 20). In these beams the strands began to slip a little already quite soon after the start of the test, while at failure the slip that had occurred in a number of strands was then approximately 25 mm. The explanation for premature failure was accordingly sought in this slip of the prestressing steel.

In order to discover the cause of slip, the ends of beam 2 were sawn through after the test. It was found that in the longitudinal direction of the beam horizontal and vertical cracks extending to the prestressing steel had developed (Fig. 21). These cracks are attributable to high internally induced tensile stresses, as explained in Chapter 2 (see Fig. 12).

In view of the above-mentioned results, the further test programme was oriented to:

Fig. 21. Internal cracking along the strands in test beam 2.

- accepting the slip of the strands and trying to improve the fire resistance by structural measures;
- trying to prevent or greatly to reduce the slip of the strands.

In connection with these considerations the following measures and arrangements were included in the research and their effect was investigated:

a. increasing the quantity of conventional reinforcement by providing stirrups (in some cases: zig-zag "stirrups") along the whole length of the beam; over a length of about 1.00 m near each end of the beam the stirrup spacing was made smaller than in the intermediate part of the beam;
b. using stirrup reinforcement in the form of fabric mesh, i.e., comprising vertical and horizontal bars;
c. using 9.6 mm diameter strands instead of 12.5 mm diameter;
d. concentrating the prestressing steel and enclosing the strands in a helix at the ends of the beam;
e. anchoring the prestressing steel at the ends, thus obviating any slip of the steel;
f. protecting the ends of the beam from direct heating for a distance of about 600 mm (cold ends);
g. the effect of the age of the concrete: the above-mentioned beams were about 2 months old at the time of testing, during which period they had been stored under normal indoor conditions; as it was considered to be just possible that slip might be affected by the moisture still present in these young beams, one beam was artificially dried before being tested.

The data relating to these beams are presented in Fig. 17 and in Table 4. The principal results are also given in that table.

To judge the effect of the various measures and arrangements it is important to know the fire resistance corresponding to the attainment of the critical temperature in the prestressing steel.

For the T 40/70 beams prestressed with 12.5 mm diameter 7-wire strands distributed over the cross-section the maximum fire resistance was attained with the beams 7, 8 and 10 (cold ends, i.e., insulated for a distance of 600 mm, or strands provided with anchorages). Failure occurred in the mid-span region of the beam after a period of 74, $82\frac{1}{2}$ and 80 minutes respectively; it was caused by a marked increase in the elongation of the prestressing steel, so that failure occurred in consequence of the critical steel temperature being reached. The fire resistance values found in these tests were in reasonably good agreement with the calculated value of 75 minutes (Section 4.3.2).

In contrast with beam 8 with cold ends 600 mm in length, in the case of beam 2, one end of which was insulated for a distance of 450 mm, premature slip of the prestressing steel did occur. The slip at this insulated end was just as large as the slip at the other end, which had only 50 mm insulation. Failure occurred at the *cold* end, however, which means that the *critical* condition had been attained there too.

It would appear that adequate bond is ensured if the ends of the beam are kept "cold", i.e., heat-insulated, for a distance of at least 600 mm, which approximately corresponds to the transmission length.

In the case of the beams which were exposed to fire all along their length and which were provided with stirrup reinforcement (whether or not of the zig-zag type) this fire resistance was not attained (beams 4, 5, 6, 13 and 15). On the other hand, the anticipated fire resistance was attained by the beams in which fabric reinforcement had been used in combination with end meshes (anti-splitting) or stirrups (beams 9 and 14).

3.3 Effect of stirrup reinforcement with regard to 12.5 mm strands developing slip

For further studying the effect of the (conventional) reinforcing steel, in Fig. 22 the total force which, at the fire resistance determined, can be resisted in the shear force region of the beam (this region is 3.40 m long) has been plotted against the measured fire resistance. The results of the calculation of the reduced forces for the test beams with slipped strands are given in Table 5.

In stating the quantity of stirrup reinforcement the four end meshes have in all cases been ignored. These sets of mesh reinforcement do indeed perform a useful function, but make little or no contribution to the shear resisting capacity of the beam because they are located over the bearings.
It appears from Fig. 22 that in the region under investigation the fire resistance of the beams increases in a practically linear manner with the magnitude of the vertically resistable force corresponding to the fire resistance attained. The slip of the prestressing steel undergoes no, or hardly any, reduction, however. With reference to the beam with zig-zag stirrup reinforcement it should be noted that this "stirrup" does not extend into the compressive zone of the concrete. Because of this, fairly large

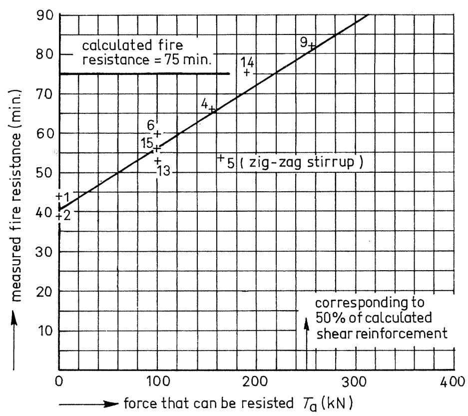


Fig. 22. Relationship between fire resistance and the force that can be resisted by stirrup reinforcement for test beams T 40/70 with "hot heads", non-concentrated and non-anchored 12.5 mm 7-wire prestressing strands.

Table 5. Stirrup reinforcement in shear force region and force to be resisted by this reinforcement

test beam	stirrup reinforcement*	A_v (mm²)	grade of steel	measured fire resistance (min)	T_a at normal temperature (kN)	reduction factor	T_a reduced (kN) ***
1	–	0	–	44	0		0
2	–	0	–	39	0		0
4	28 ∅ 6	792	FeB 220	$66\frac{1}{2}$	175	0.88	155
5	29 ∅ 6	750**	FeB 220	$53\frac{1}{2}$	165	1.0	165
6	16 ∅ 6	452	FeB 220	60	100	1.0	100
9	22 ∅ 6	622	FeB 220	82	135 } 405	0.66	90 } 255
	34 ∅ 4.5	541	FeB 500		270 }	0.61	165 }
13	16 ∅ 6	452	FeB 220	53	100	1.0	100
14	34 ∅ 4.5	541	FeB 500	$75\frac{1}{2}$	270	0.70	190
15	16 ∅ 6	452	FeB 220	56	100	1.0	100

* end mesh reinforcements (10 mm bars) neglected
** vertical component
*** forces reduced because of the lowered yield stress in the stirrup reinforcement on attainment of the fire resistance, based on heating-up rate of 6 °C per minute (see Fig. 24)

cracks developed at the junction of the web and flange in this beam; this is in fact an undesirable situation.

A closer examination and interpretation of the results presented in Fig. 22 are necessary in order to ascertain what stirrup reinforcement is needed in order to prevent a reduction in fire resistance as a result of slip.

First, it will be considered how much stirrup reinforcement is required to enable the shear force acting during the fire test (average approx. 37 kN) to be wholly resisted by reinforcement. In that case, according to the method set forth in Chapter 4, and taking account of the reduction in the material strengths due to the high temperature, stirrup reinforcement with a steel sectional area of 840 mm²/lin.m (see Section 4.4.2) should be provided, which corresponds to 5.2‰ of the horizontal sectional area of the web.

The total force T_a that this stirrup reinforcement can resist in the shear force region of the beam on attainment of its fire resistance (stirrup temperature 450 °C) is 510 kN (see Section 4.4.3).

From Fig. 22 it follows that for reaching the critical steel temperature at the calculated fire resistance of 75 minutes a smaller quantity of stirrup reinforcement would suffice, namely, about 50% of the calculated shear reinforcement. In the lattice analogy adopted here, the "bottom chord" of the lattice system is of major importance in addition to the vertical stirrup reinforcement.

If 50% of the calculated shear reinforcement is installed, the beam will be found to attain a fire resistance at which the critical temperature is also reached in the prestressing steel. At failure due to fire the stress in the prestressing steel will be (see Appendix A):

$$\sigma_a = \frac{\text{tensile strength of the steel}}{\text{margin with regard to failure}} = \frac{1860}{2.47} = 755 \text{ N/mm}^2$$

If this stress – with slipping strands – is to be developed by bond (= friction) to the concrete, it is found that the bond stress τ should be at least (on the assumption that τ = constant):

$$\tau = A_p \sigma_a / \pi \phi l' = \frac{93 \times 755}{\pi \times 12.5 \times 3.5 \times 10^3} = 0.51 \text{ N/mm}^2$$

where:

A_p = cross-sectional area of a 12.5 mm 7-wire strand = 93 mm^2;
l' = length of the shear force region plus the bearing length = $(3.4+0.1)\ 10^3$ mm.

From the following considerations it emerges that the bond stress cannot have been much higher: In the test beams 9 and 14 an amount of slip of the order of 15 mm was found at the end of the test (see Table 4). The strain in the steel due to the prestress was approximately 6‰ for a steel stress of 1.24 kN/mm^2, corresponding to an elongation of 24 mm along half the length of the beam. After the test there was a residual elongation of 9 mm. This gives a steel stress of the following magnitude at the mid-span section of the beam (τ = constant, therefore triangular stress distribution):

$$\sigma_a = 9 \times 2 \times 1.24/24 = 0.93 \text{ kN/mm}^2$$

The bond stress is therefore:

$$\tau = \frac{93 \times 930}{\pi \times 12.5 \times 4 \times 10^3} = 0.55 \text{ N/mm}^2$$

It can therefore be supposed that the bond stress was only little more than 0.5 N/mm^2. Since the bond stress can vary greatly (as the slip measurements have shown), a value not exceeding 0.5 N/mm^2 should be introduced for the bond stress τ in analysing the strength of the "bottom chord".

3.4 Effect of 9.6 mm instead of 12.5 mm strands

In beams in which strands of 9.6 mm diameter were used the amount of slip of these tendons was found to be much less than that of the 12.5 mm strands in the beams discussed in the foregoing. In beams 11 and 17 (see Table 4) this slip in the 9.6 mm strands was about 30% of that found in 12.5 mm strands distributed through the cross-section of the beam. Beam 17 attained a fire resistance corresponding to the calculated value. The measured fire resistance was 95 minutes. The calculation (see Chapter 4) based on a concrete cross-sectional area of 72000 mm^2 and a concrete cover of about 60 mm gives $B = 450/5 = 90$ minutes. The bond developed by 9.6 mm strands is of course better than that developed by 12.5 mm strands. Unfortunately, from this investigation it cannot be inferred that for a required fire resistance in excess of 90 minutes the use of 9.6 mm strands will necessarily also lead to favourable results.

3.5 Effect of helical reinforcement

Reduction of slip, and a fire resistance corresponding to the attainment of the critical steel temperature, were likewise found in the beams in which the 12.5 mm strands were installed in a concentrated arrangement (beams 12, 16 and 18, see Table 4 and Fig. 17). The vertical reinforcement consisted of ordinary stirrups or a zig-zag "stirrup" in the web, and the ends of the bundled strands were enclosed within a helical reinforcement.

According to CUR Report 13 (and Chapter 4), on the basis of a concrete cross-sectional area of 72000 mm^2 and a concrete cover of 70 mm, these beams can be expected to attain a fire resistance of 105 minutes; the actual values found are, respectively, 102, 117 and 109 minutes.

All the helixes employed were of grade FeB 220 reinforcing steel, with a pitch of 60 mm and an outside diameter of 85 mm. The diameter of the helical bars and the length of the helixes varied: in beam 12 the helix consisted of a 10 mm diameter bar and was 1 m long; in beams 16 and 18 the helix consisted of 6 mm diameter bar and was 0.6 m and 1 m long, respectively.

In all the beams provided with helixes the prestressing steel reached the critical temperature. It is not possible to select from these test results ‹ ne particular helix that can be said to have performed better than the others. There were found to be fairly large differences in slip among the various beams that had been provided with helixes:

- in beam 16 there was an average slip of 0.3 mm/5 minutes (i.e., approximately the same effect as when 9.6 mm strands distributed through the cross-section are employed);
- in beam 18 there was an average slip of 0.8 mm/5 minutes.

The effect of the helixes in the given circumstances is not very clear, so that it is not possible to give a generally-valid method of design for them. It is understandable, however, that the bond developed by the strands is improved by the presence of the helix. Besides, the latter probably has the effect of deflecting the internal cracks around the helically bound core region and thus away from the strands themselves, which of course also has a favourable effect.

As already stated, it is hardly practicable to determine the dimensions of the helical reinforcement by calculation. For beams prestressed with seven or fewer concentrated 12.5 mm diameter strands it is recommended to use a helix 1 m in length and consisting of a 6 mm or 10 mm diameter bar. If the number of strands in the beam is larger, they should be disposed in two or even more concentrated groups.

3.6 Moisture content

It has already been mentioned that, at the time of testing, the concrete was still quite young and thus had a fairly high moisture content; the beams were only about two months old. In the case of beam 15, however, artificial drying ensured that its moisture content was about the same as that of beams which had been stored for $1\frac{1}{2}$ years under "indoor" conditions at 20 °C and 65% relatively humidity.

Nevertheless, no significant differences in slip of the strands or in fire resistance were found to occur between this beam and comparable "moist" ones.

3.7 Conclusions

With regard to the beams prestressed with 12.5 mm pretensioned strands and exposed in their entirety to fire it can be stated that the critical steel temperature is not reached unless:

a. Slip of the pretensioned strands is prevented. This can be achieved by the following precautions:
   - anchoring the strands individually;
   - keeping the ends of the beams cold or heat-insulated for a distance equal to 1.5 times the bond length, i.e., a length of 1 m.
b. Installing stirrup reinforcement capable of resisting half the shear force occurring during the fire; this reinforcement should extend into the compressive zone of the beam. The calculation of the stirrups is presented in Chapter 4. Such stirrup reinforcement, if installed, will not prevent or reduce slip of 12.5 mm strands, however. The tests show that in the beams in question the bond strength developed was approximately 0.5 N/mm^2. This is of major importance with regard to the strength of the "bottom chord" of the analogous truss or lattice system. In view of the considerable scatter that the bond stress is likely to display, it is advisable to assume a bond stress not exceeding 0.5 N/mm^2 in analysing the strength of this "bottom chord".
c. Improving the bond of the strands. In the investigations reported here this was achieved in two ways:
   - using small-diameter strands: with 9.6 mm strands the desired fire resistance was attained; in this case there is still some slip of the strands, but much less than with 12.5 mm strands;
   - adopting a concentrated arrangement for 12.5 mm strands, with a helical reinforcement around their ends; one such concentrated group should comprise not more than seven 12.5 mm strands; a length of 1 m is recommended for the helix.

For beams which are substantially similar to those investigated in the research reported here, the above-mentioned measures provide a good basis for their design in a case where approximately 90 minutes fire resistance is required.

CHAPTER 4

CALCULATION OF THE FIRE RESISTANCE OF PRESTRESSED CONCRETE BEAMS

4.1 General considerations

In this chapter some theoretical considerations will be presented which will be checked for their validity against the test results reported in the preceding chapters.

The research on the fire resistance of prestressed concrete beams indicates that two types of failure are liable to occur as a result of a fire:

- bending moment failure, caused by the critical temperature being reached in the prestressing steel;
- shear failure, which occurs in consequence of a continuous horizontal crack developing (between the compressive and the tensile zone of the beam) or of slip of the prestressing strands.

The first form of failure usually occurs in beams of which the ends remain cold or in which the prestressing steel is anchored to obviate slip. If such beams are of I-shaped cross-section, in about 10% of them there occurs such horizontal cracking that the shear force cannot be transmitted by the *erratic* cracks that develop. As a result, these beams fail prematurely unless they have been designed to withstand shear-type failure.

In the earlier research (CUR Report 13) the last-mentioned form of failure had not been recognized for what it was. The beams that had failure prematurely were regarded as showing freakish behaviour and accordingly called "rogues".

Unless the conditions stated in Section 3.7 are satisfied, shear-type failure under fire conditions will also occur in prestressed concrete beams provided with 7-wire 12.5 mm pretensioned strands and exposed to the fire along the entire length. This is because in that case slip of the strands occurs. Insulation of the ends of the beam, or suitable measures to improve the bond of the strands to the concrete, can obviate this phenomenon. Examples of suitable measures are: using strands of smaller diameter, using strands with indented instead of plain wires, adopting a concentrated arrangement for the strands, and enclosing them within a helical reinforcement. Of course, slip is entirely prevented by providing the strands with end anchorages.
In order to ensure the requisite fire resistance, the beams must therefore be designed to withstand one or both of the above-mentioned forms of failure. The methods available for such design will be summarized here.

4.2 Limit state of fire

The fire resistance of a structural member (or of a structure as a whole) is the time during which this member (or structure) can be exposed to a fire test conforming to the standard time-temperature curve, while subjected to the total design loading, before failure occurs. These conditions are defined in Netherlands Standard NEN 1076, Chapter D "Fire resistance of structures".

The behaviour of the structure can be assessed in the following ways:

a. Basing the temperature distribution in the structure on the standard time-temperature curve (see Fig. 1) and the required fire resistance. The heating-up rate indicated in Fig. 23 (from CUR Report 13) may, for example, be adopted for determining the temperature. In general, this temperature distribution can also be determined by calculation.

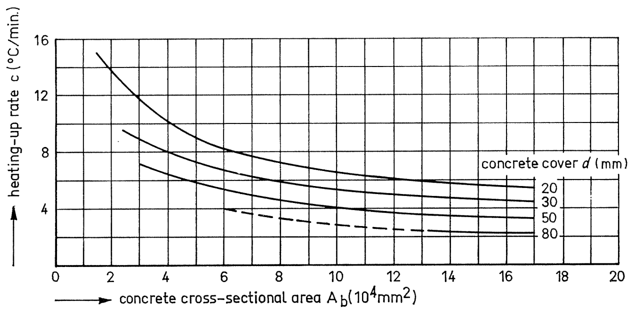


Fig. 23. Heating-up rate of steel in concrete: valid for the standard time-temperature curve and a fire resistance of approximately 100 minutes.

b. Determining the material properties from the design values (valid for normal temperature) and reducing these in accordance with the temperature distribution envisaged in a (see Figs. 24 and 25).

4.3 Fire resistance with regard to bending moment failure (attainment of critical temperature in prestressing steel)

In the case under consideration, failure occurs because the temperature of the steel becomes too high, so that the strains in the steel increase considerably. This results in collapse, because equilibrium with the moment due to external loading is then no longer possible.
Calculation of the fire resistance can be done in either of two ways:

- With regard to the limit state of fire. This can be done by the introduction of suitably modified or adjusted material properties (taking due account of the temperature expected to occur in the materials on reaching the required fire resistance period) and calculating the failure moment or the required concrete cover or the required reinforcement in very much the usual way, i.e., as for normal temperature.

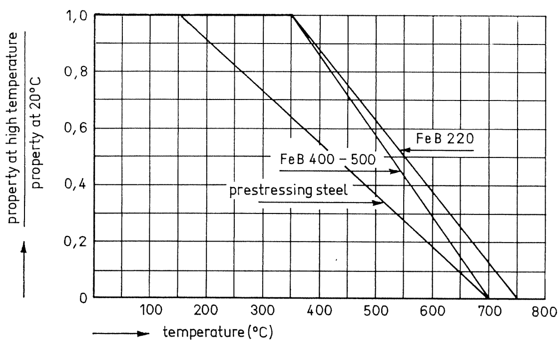


Fig. 24. Schematized relationship between the tensile strength of prestressing steel, the yield stress of reinforcing steel and the steel temperature.

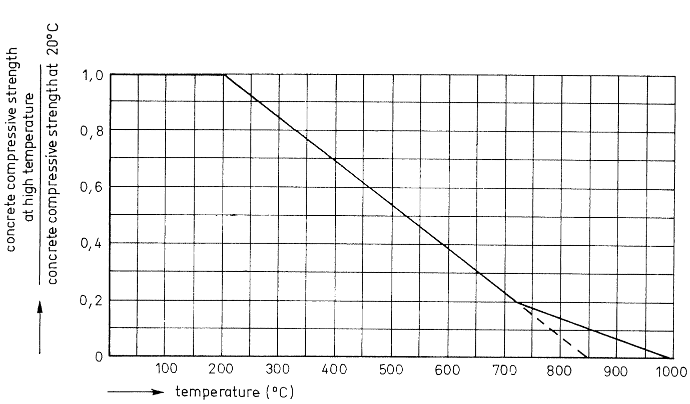


Fig. 25. Schematized relationship between concrete compressive strength and concrete temperature.

– In accordance with a simplified procedure developed in CUR Report 13, using an empirically determined heating-up rate c (°C/min.) of the prestressing steel which is a function of the concrete cover d* to the steel in question and of the cross-sectional area A_b of the concrete in which that steel is embedded (see Fig. 23). Furthermore a critical temperature T_{kr} has – likewise empirically – been established for the prestressing steel, i.e., the temperature at which the commonly employed prestressing steels in which normal working stresses occur (under total design load) are no longer able to develop the forces necessary to maintain equilibrium, so that the beam collapses. This temperature is 450 °C. According to this procedure the fire resistance B with regard to the attainment of the critical steel temperature is expressed (in minutes) by:

$$B = T_{kr}/c$$

* This concrete cover is not the cover c as defined in the VB-1974, Netherlands code of practice, and for this reason a different notation has been adopted.

4.3.1 *Fire resistance of I-beams*

For the rogue beams of CUR Report 13 (full-size beams) considered in the present report:

- concrete cross-sectional area $A_b = 70400\ \text{mm}^2$;
- concrete cover $d = 30$ mm.

According to Fig. 23 the heating-up rate $c = 6\,°\text{C/min}$, so that at the critical steel temperature of 450 °C the fire resistance is:

$$B = 450/6 = 75\ \text{min}$$

For the model beams discussed in Chapter 2, with a scale factor of 1.5 in relation to the full-size (prototype) beams, the fire resistance on attainment of the critical steel temperature is:

$$B_{\text{model}} = 75/1.5^2 = 33.3\ \text{min}$$

4.3.2 *Fire resistance of T-elements*

The data for the T-elements envisaged in Chapter 3 are:

- concrete cross-sectional area $A_b = 72000\ \text{mm}^2$ (without the flange);
- concrete cover $d = 38$ mm;
- heating-up rate $c \approx 6\,°\text{C/min}$ (according to Fig. 23).

The fire resistance of the T-elements is therefore:

$$B = 450/6 = 75\ \text{min}$$

4.4 Fire resistance with regard to shear failure

Calculating the fire resistance with regard to shear failure is in fact no different from an analysis of shear for the ultimate limit state at normal temperature.

In principle, the calculation may be based on one of the following analogies:

a. a truss or lattice girder;
b. an arch with tie-member.

With these approximations a maximum quantity of stirrup reinforcement is obtained (under certain circumstances) or no stirrups at all, respectively. In general, in view of the stirrup reinforcement it requires, the lattice analogy is a safe assumption. Shear analysis in connection with the fire resistance of prestressed concrete beams has been found necessary because a continuous horizontal crack occurs and/or the prestressing strands slip.

More particularly in the latter case the assumption of an arch with tie-member is incorrect. For this reason only the lattice (or truss) analogy will be considered here.

It is presupposed that – in the stage in which the structure is analysed – the prestressing force has a negligible effect on the slope of the diagonal compression members (struts) of the lattice system. In general, it is considered sufficiently accurate to assume the struts to be inclined at 45° with respect to the centre-line of the beam. The equations are derived for the general case.

With vertical stirrups, the following expression is obtained from equilibrium considerations (see Fig. 26):

$$T_u = \omega_t f_{et} b_0 z \cot\alpha \tag{1}$$

where:

ω_t = reinforcement proportion of a stirrup = A_t/tb_0
A_t = cross-sectional area of a stirrup (two bars)
t = stirrup spacing
b_0 = thickness of web
f_{et} = yield stress of the stirrups at the temperature in the stirrups on attainment of fire resistance
z = internal lever arm $\approx h$

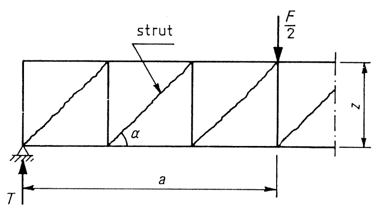


Fig. 26. Diagram of the lattice system adopted in the shear analysis.

The stress then occurring in the struts is:

$$\sigma_b' = \frac{2T_u}{b_0 z}\,\frac{1}{\sin 2\alpha} \tag{2}$$

In order to judge whether the stirrup reinforcement is indeed determinative, it must be checked whether:

$$\sigma_b' \leqq f_{bt}'$$

where:

f_{bt}' = design value of the compressive strength of the concrete at the temperature corresponding to the fire resistance

For the tensile force in the bottom chord of the lattice system:

$$N_a = T_u a/z \tag{3}$$

On substitution of equation (1) into equation (3) the following expression is found:

$$N_a = A_t f_{et}(a/t)\cot\alpha$$

Hence it appears that the capacity of the stirrup reinforcement in a region "a" – reckoned from the bearing – should be at least equal to the tensile force in the longitudinal steel due to the largest bending moment in that region, divided by cot α.

If one of the two – longitudinal steel or stirrup reinforcement – has been relatively overdesigned, then the other is determinative with regard to the loadbearing capacity of the beam.

4.4.1 *Shear analysis for fire in the case of "horizontal" cracking*

The model beams discussed in Chapter 2 (see Fig. 8) have a fire resistance of 33.3 minutes on the basis of the critical temperature of 450°C being attained in the prestressing steel (Section 4.3.1). Neglecting the difference in the concrete cover to the stirrups and to the prestressing steel respectively, this means of course that the temperature in the stirrup steel will also be 450°C after 33.3 minutes. From Fig. 14 it follows that the yield stress in the stirrups, which is 300 N/mm^2 at normal temperature,* is subject to a reduction factor of 0.75. The shear force T_u occurring during the fire test was 20 kN. Furthermore: $b_0 = 53$ mm and $z \approx h = 220$ mm. The stirrup reinforcement required to ensure that the above-mentioned fire resistance, on the assumption that the struts of the lattice system are inclined at 45°, is duly attained amounts to:

$$\omega_t = \frac{T_u}{f_{et} b_0 z} = \frac{20000}{0.75 \times 300 \times 53 \times 220} \times 10^3 = 7.6‰$$

Stirrups of 4 mm diameter at 60 mm spacing are required.

In calculating the shear capacity under fire conditions the proportion assignable to uncracked concrete has been taken as $T_0 = 0$.

If the average concrete temperature is assumed to be equal to the temperature in the stirrup reinforcement – which is a safe assumption – then, according to Fig. 25, on attainment of the fire resistance there will, for an average cube strength $f'_{cm} = 65$ N/mm^2, still be an average compressive strength $f'_{bm} = 0.6 \times (0.8 \times 65) = 31$ N/mm^2 available. The compressive stress in the concrete of the struts of the lattice system is therefore:

$$\sigma'_b = \frac{2T_u}{b_0 z} = \frac{2 \times 20000}{53 \times 220} = 3.4 \text{ N/mm}^2$$

The struts are therefore not determinative with regard to the shear capacity of the beams under consideration.

In this example the strength of the bottom chord presents no problems, since the beams have cold heads or anchored prestressing steel.

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* As the shear analysis presented here serves also as the interpretation of test results discussed, the measured yield stress has been used.

4.4.2 *Shear analysis for fire in the case of slipping strands*

This calculation relates to the T-elements discussed in Chapter 3 (see Fig. 17 and Appendix A). During the fire test the beams were subject to shear force ranging from 43 kN at the supports to $43 - 3.4 \times 3.4 = 31.4$ kN at the point loads.

The average shear force in the region in question is therefore:

$$(43 + 31.4)/2 = 37.2 \text{ kN}$$

For these T-elements the fire resistance with regard to the attainment of the critical temperature of 450 °C in the prestressing steel is 75 minutes (see Section 4.3.2). Neglecting the difference in concrete cover to the stirrups and to the prestressing steel, it follows that the temperature in the stirrups is also 450 °C after 75 minutes.

The yield stress of the stirrup reinforcement, which is 240 N/mm^2* at normal temperature, must be reduced to 0.75×240 N/mm^2 (see Fig. 24).

The requisite stirrup reinforcement for $z \approx h = 400 - 154 = 246$ mm, with struts inclined at 45°, is expressed by:

$$A_t = \frac{T_u}{f_{et}z} = \frac{37200}{0.75 \times 240 \times 246} \times 10^3 = 840 \text{ mm}^2/\text{lin.m}$$

This cross-sectional area of stirrup reinforcement per meter length of the beam corresponds to 5.2‰ of the horizontal sectional area of the web (160 mm web thickness).

The total reduced tensile force that can be resisted in the shear region with a length of 3.40 m is therefore:

$$0.75 \times 240 \times 840 \times 3.40 = 510 \text{ kN}$$

The fire tests show that half the shear reinforcement, as calculated here, is sufficient to enable the fire resistance of 75 minutes to be attained.

On the safe assumption that the average temperature of the concrete is the same as that of the stirrups, it follows that the compressive strength of the concrete at 450 °C is, according to Fig. 25, equal to 0.6 × the compressive strength at 20 °C. The available average concrete compressive strength for $f'_{cm} = 45$ N/mm^2, is therefore $0.6 \times 0.8 \times 45 = 21.6$ N/mm^2. The concrete stress in the struts is then:

$$\sigma'_b = \frac{2T_u}{b_0 z} = \frac{2 \times 37200}{160 \times 246} = 1.9 \text{ N/mm}^2$$

The struts (compression members in the lattice system) are therefore not determinative.

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* As the shear analysis presented here serves also as the interpretation of test results discussed, the measured yield stress has been used.

Finally, it must be checked whether the bottom chord of the lattice system possesses adequate strength. At the point loads the stress in the prestressing steel at the instant of failure is:

$$\sigma_a = 1860/2.47 = 755 \text{ N/mm}^2$$

If the bond stress is taken as $\tau = 0.5$ N/mm^2, the steel stress at the point loads is determinable from:

$$0.5 \times \pi \times 12.5 \times 3.5 \times 10^3 = 93\sigma_a$$

$$\sigma_a = 740 \text{ N/mm}^2$$

It can be inferred that the tensile strength in the bottom chord is likewise attained at the required fire resistance.

Appendix A

CALCULATION OF THE LOAD THAT MUST BE PRESENT ON THE T-ELEMENTS DURING THE FIRE TEST

The following data were obtained from the design calculations of the manufacturer of the beams (Nederlandse Spanbeton Maatschappij B.V., Alphen aan de Rijn):

- permissible bending moment under working load: 127.0 kNm;
- failure moment: 313.8 kNm; therefore $\gamma = 313.8/127 = 2.47$;
- average compressive strength of concrete: $f'_{cm} = 45$ N/mm^2;
- steel grade: FeP 1860;
- dead weight of the beam including topping: 3.40 kN/lin.m;
- variable load: $0.7 \times 4.00 = 2.80$ kN/lin.m

From these data the span l adopted in actual practice can be calculated:

$$\tfrac{1}{8}(3.40+2.80)l^2 = 127.0$$

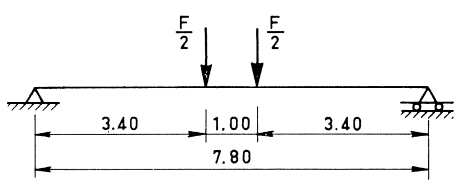
hence

$$l = 12.80 \text{ m}$$

The maximum shear force T is then:

$$T = \frac{12.80}{2}(3.40+2.80) = 39.7 \text{ kN}$$

Because of the length of the test furnace, the span adopted in the fire test must not exceed 7.80 m. In order to obtain the best approximation for the bending moment of 127.0 kNm as well as for the shear force of 39.7 kN with this shorter span, two point loads $F/2$ at 3.40 m from the bearings were applied to the beam.



To allow sufficient space for the ballast, the distance between the point loads had to be at least 1 m.

To produce the required bending moment the point loads had to be of the following magnitude:

$$\tfrac{1}{8} \times 3.40 \times 7.80^2 + \frac{F}{2} \times 3.40 = 127.0$$

hence

$$\frac{F}{2} = 29.75 \text{ kN}$$

The shear force developed in the beam under this load is:

$$T = \frac{7.80}{2} \times 3.40 + 29.75 = 43.0 \text{ kN}$$

The shear force that occurred during the fire test was therefore about 3 kN larger than the shear force in actual practice.