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FIRE RESISTANCE OF JOINT DETAILS IN LOADBEARING TIMBER CONSTRUCTION - A LITERATURE SURVEY

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PREFACE

This translation, by Barbro Harris (Victoria University of Wellington) and P K A Yiu (Building Research Association of New Zealand) of Olle Carling's paper Brandmotstånd hos infästningdetaljer och Förband i bärande trakonstruktioner, was undertaken to provide a summary of recent work in West Germany and Scandinavia on the fire resistance of joint details in heavy timber construction, e.g., glulam timber beams and columns. BRANZ gratefully acknowledges the cooperation of the author, Olle Carling, and Professor Kai Ödeen, of the Building Materials Group, Royal Institute of Technology, Sweden, in the production of this paper.

Note: This translation has been made in good faith but may be subject to error. Any apparent anomalies noted in the original text have been brought to the reader's attention by the interpolation of notes in square brackets.

This report is intended for fire engineers, design engineers, architects and research workers.

FIRE RESISTANCE OF JOINT DETAILS IN LOADBEARING TIMBER CONSTRUCTION - A LITERATURE SURVEY

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REFERENCE

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KEYWORDS

From Construction Industry Thesaurus - BRANZ edition: Bolts; Charring; Fire Protection; Fire Resistance; Fire Tests; Gaps; Gusset Plates; Joints; Loadbearing; Nails; Timber; Tooth Plate Connectors; Wood Laminates.

ABSTRACT

This literature survey of fire resistance of joint details in load-bearing timber construction presents a summary of work done on the behaviour of joints and joint details in West Germany, Denmark, Sweden and Norway.

CONTENTS

0. INTRODUCTION

For the fire design of different elements in heavy timber construction, e.g., glulam timber beams and columns, the fire resistance is usually determined by an analytical solution. From existing knowledge, this calculation is based on largely simplified assumptions with respect to the growth of the charred zone as well as the physical and mechanical properties of the timber during fire. Despite the lack of comprehensive information, the results produced are still useful in practical terms.

An estimation of the load capacity of composite construction during fire also includes an analysis of the joint details. At present, there is no method for calculating the behaviour of these components nor their load capacity during a fire. Even the data from fire exposure, e.g., in the form of standard fire tests, are either very poor or insufficient.

In the last few years, a certain amount of work has been done in this area in West Germany, Finland, Denmark, Sweden and Norway. This report presents a summary of the available information and it also forms the basis of a proposal for a programme of continuing Swedish research on this subject. The work was financed by the Timber Technology Centre, Steering Group 21 : Glulam Timber and Fire Research.

1. MECHANICAL AND THERMAL PROPERTIES OF MATERIAL AT ELEVATED TEMPERATURES

In order to assess the behaviour and load-bearing capacity of composite construction during fire, information on the stiffness and strength of the material involved at elevated temperatures is necessary. Also the thermal properties, e.g., conductivity, are of great importance. However, the level of knowledge varies considerably for different materials.

1.1 Steel

The determination of the load capacity of joint details in mild steel is based on a 0.2-limit, i.e., the stress which gives a residual strain of 0.2%; because a well defined yield point does not exist at elevated temperatures.

Magnusson et al [1.1] as well as Pettersson and Odeen [1.2] show in Figure 1.1 the relationship between the 0.2-limit and the modulus of elasticity with the steel temperature.

Thor [1.3], [1.4] has presented a method for calculating deformations which includes the creep characteristics of steel at high temperatures.

References [1.1] and [1.2] contain information on the various thermal properties of steel, e.g., heat capacity and conductivity, and their variations with temperatures.

1.2 Timber

Available information on the mechanical properties of timber is largely based on investigations carried out on small clear test samples at constant temperature and moisture content, both of which will vary with time in a real fire. It has been shown that changes in both temperature and moisture conditions during loading increase the shrinkage of the timber, a phenomenon which has been assumed to be closely related to the fracture mechanism of the material under consideration [1.5], [1.6].

In the studies of joint details, the shear strength in different directions as well as the tensile strength are of primary interest, especially in the temperature range of 200-280°C, due to the effects of the pyrolosis zone. Load transmission, as a rule, takes place through bearing or shear.

Investigations into the strength of timber loaded parallel to the grain direction at elevated temperatures have been described by Kollman [1.7] (but only up to 100°C), Schaffer [1.8], Knudsen and Schniewind [1.9], Kallioniemi [1.10] and Gerhards [1.11].

Kallioniemi expressed his results as a regression relationship between the strength (which is expressed as a percentage of the value at 20°C and 12% moisture content), the temperature T (°C) and the moisture content u (%). The investigation was carried out using small clear samples of spruce and pine and each curve was based on 144 tests.

spruce : $f_c/f_{c,20,12} = 208 - 21.3 \ln T - 3.7 u$ (%)

pine : $f_c/f_{c,20,12} = 233 - 28.7 \ln T - 3.9 u$ (%)

In Figure 1.2, results from references [1.7] to [1.10] are also expressed as a percentage of the compressive strength at 20°C.

These results show large variations both within the same investigation and between them. This is due to difficulties in the measurements, the different types of timber which were investigated, as well as variation in quality which is a characteristic of all timber materials.

Kallioniemi [1.10] has also investigated the relationships between shear strength, temperature and moisture content and presented the results in the same way as for the compressive strength.

spruce : $f_v/f_{v,20,12} = 128 - 0.35 T - 1.8 u$ (%)

pine : $f_v/f_{v,20,12} = 126 - 0.31 T - 1.7 u$ (%)

In Figure 1.3, the relationships are illustrated in graph form. The lower curve was determined on the basis of measurements in a glulam timber beam in the same test. The moisture content varied from 5%, 5 mm inside the pyrolysis zone, to 12% in those parts which were situated more than 20 mm away from this zone. The temperature decreased from 230°C in the pyrolysis zone to 20°C in the central part of the beam.

Knudsen and Schniewind [1.9] as well as Kallioniemi have investigated the residual strength after cooling and reconditioning, see Figure 1.2. Kallioniemi indicated that the compressive strength after cooling was unchanged whereas the shear strength was reduced by 10%. However, in their results on compressive strength, the effects of loading at an angle to the grain direction or at the edge of an opening were not included.

A summary of the different results from work on the thermal properties of timber at various temperature and moisture content has been given by Jonsson and Pettersson [1.12] and also by Hadvig [1.13].

1.3 Charcoal

There is very little published information on the mechanical properties of charcoal. However, Aarnio and Kallioniemi [1.14], [1.15] have studied the compressive strength of charcoal layers 13 mm thick and found a mean value of 0.30 MPa from 15 tests. The standard deviation assuming a normal distribution was 0.09 MPa. Based on these figures, the characteristic strength can be shown to be 0.12 MPa. The charcoal layer was compressed by an average of 6% during the tests.

Information on the thermal properties of charcoal from various sources has been gathered by Jonsson and Pettersson [1.12] and Hadvig [1.13]. All the information refers to unloaded charcoal, that on loaded charcoal was not available.

[1.1] Magnusson, S. E., Pettersson, O. and Thor, J. 1974. Fire design of steel structures. Steel Construction Institute, Publication No. 38. Stockholm.

[1.2] Pettersson, O. and Odeen, K. 1978. Fire design, principle, data, examples. Liber Publishers. Stockholm.

[1.3] Thor, J. 1972. Calculation of fire effects on the statically determinate and statically indeterminate deformations of steel beams and their critical loads. Steel Building Institute, Report 22:9. Stockholm.

[1.4] Thor, J. 1972. Investigation of creep behaviour of different types of steel construction during fire. Steel Research Association, D 40. Stockholm.

[1.5] Kitahara, K. and Yukawa, K. 1964. The influence of the change of temperature on creep in bending. Journal of Japan Wood Research Society, 10(5): 169-175.

[1.6] Armstrong, L. D. and Kingston, R. S. 1960. Effect of moisture changes on creep in wood. Nature, 185(4716): 862-863. London.

[1.7] Kollman, F. 1951. Technology of timber and timber materials. Springer Verlag. Berlin.

[1.8] Schaffer, E. 1971. Elevated temperature effect on the longitudinal mechanical properties of wood. University of Wisconsin, PhD Thesis. Madison.

[1.9] Knudsen, R.M. and Schniewind, A. P. 1975. Performance of structural wood members exposed to fire. Forest Products Journal, 25(2): 23-32. Madison.

[1.10] Kallioniemi, P. 1980. The strength of wood structures during fires. Symposium 9: Fire Resistance of Wood Structures. Technical Research Centre of Finland (VTT). Esbo.

[1.11] Gerhards, C. C. 1982. Effects of moisture content and temperature on the mechanical properties of wood: an analysis of immediate effects. Wood and Fiber, No. 1.

[1.12] Jonsson, R. and Pettersson, O. 1983. Trakonstruktioner och brand -kunskapsoversikt och forskningsbehov (in Swedish, title translates to - Timber structure and fire - review of knowledge and need for new research). Lund Institute of Technology, Report LUTVDG/(TVBB 3015). Lund.

[1.13] Hadvig, S. 1981. Charring of timber in building fires. Technical University of Denmark, Laboratory of Heating and Air-conditioning. Lyngby.

[1.14] Aarnio, M. 1979. Liimapuurakenteiden palonkestavyydestaliitokset (Limtrakonstruktioners brandmotstandsformaga - fogarna) (in Finnish, title translates to - Glulam timber construction and the fire resistance properties of the joints). Helsinki School of Technology, Division of Building Engineering, Diploma Work. Otnas.

[1.15] Aarnio, M. and Kallioniemi, P. 1983. Kantavien puurakenteiden liitoksen palonkestavyys (Brandsakerhet hos fogar i barande trakonstruktioner) (in Finnish, title translates to - Fire Safety in joints of loadbearing timber construction). Technical Research Centre of Finland (VTT), Fire Technology Laboratory, Research Report No. 233. Esbo.

2 PYROLYSIS MODEL

Available methods for calculating the penetration of the charred zone as well as the temperature and moisture conditions at different parts of a timber construction during fire are the basis for different aspects of fire design of loadbearing timber structures.

The development of an analytical model which includes all the complex physical and chemical processes which take place when the timber gradually breaks down in a fire is almost impossible. Perhaps such a task is also meaningless because of insufficient knowledge of the various parameters in this area. Instead, research workers have been concentrating on approximate one-dimensional models which have been verified empirically. In references [2.1] to [2.4], brief descriptions of some approaches to the problem have been presented. To this end, the works of Fredlund [2.2] and Hadvig [2.4] are particularly interesting. The former because his models included the determination of temperature and moisture conditions in the cross-section at various times; however, the model was not verified experimentally. Hadvig presented, on the basis of a very comprehensive test programme, diagrams and formulae to calculate the penetration of the charred zone in timber exposed to real fire conditions in accordance with the Swedish Building Standards 1976:1 [2.5]. These formulae now form part of the Danish Standards [2.6] and are likely to be adopted in Sweden.

However, none of the methods for calculating the penetration of the charred zone is immediately applicable to joints because of the following:

(1) two- or three-dimensional flow, i.e., the influence of the anisotropic properties of timber

(2) the presence of foreign materials, i.e., concrete or steel, which have different thermal properties

(3) presence of gas barriers, e.g., steel plates.

[2.1] = [1.12]

[2.2] Fredlund, B. 1979. Modell for berakning av temperatur och fuktfordelning samt reducerat tvarsnitt i brandpaverkade trakonstruktioner (in Swedish, title translates to - Model for calculating the temperature and moisture distributions as well as the reduction in cross-section of timber construction during fire). Technical Institute of Lund, Institute for Building Technology, Internal Report IR 79-2. Lund.

[2.3] Fredlund, B. 1979. Traets antandnings - och forbranningsmekanism (in Swedish, title translates to - The flammability and combustion mechanisms of timber). Technical Institute of Lund, Institute for Building Technology, Internal Report IR 79-3. Lund.

[2.4] = [1.3]

[2.5] State Planning Commission. 1976. Brandteknisk dimensionering (in Swedish, title translates to - Fire design). Commentary on Swedish Standard 1976:1. Liber Publisher. Stockholm.

[2.6] DS 413. 1982. Dansk Ingeniorforenings norm for traekonstruktioner (in Danish, title translates to: Danish Engineering Association Standards for timber structures). Technical Publisher. Copenhagen.

3 THE EFFECTS OF GAPS AND OPENINGS IN THE JOINTS

Loadbearing timber constructions are often protected from the effects of direct fire on one or several sides, e.g., an adjoining wall or roof. The effectiveness of this protection, besides depending on the fire properties of the protecting construction, also depends to a large extent on how tightly the timber is connected to the protection. Even very small gaps can give inferior protection. It is also well known that gaps between timber members can reduce their fire resistance drastically.

Aarnio [3.1], [3.2] has investigated the effects of different gap widths (between two glulam timber beams and between a glulam timber beam and a lightweight concrete slab) on the charred zone in the vicinity of the gap during a standard fire, see Figure 3.1. Table 3.1 shows the measured penetration of the charred zone for different widths of the gap.

Aarnio found that the gap width of 5 mm was a critical figure. When the gap was wider, the beam turned into charcoal right across the gap and the corners were rounded off. The same critical figure was obtained both for a gap between a glulam timber beam and lightweight concrete as well as that between two glulam timber beams, regardless of the width of the beam and the duration of the test.

In this regard, it can be noted that Kordina and Meyer-Ottens [3.3], [3.4] gave as a condition for fire classification of joint type BMF that the gaps between the end of a beam and another beam, or the side of a column, must be less than 3 mm, see Figure 8.20.

Aarnio, in his investigations, has also measured the temperature in the gaps. From these results, it is clear that the temperature is higher in wider gaps than in narrower gaps; and the difference is most pronounced during the first 15-20 minutes of testing. After 15 minutes, the temperature in gaps less than 5 mm wide (total 12 tests) had not in any case increased above 160°C, whereas the temperature in gaps wider than 5 mm was higher than 300°C in most cases (9 out of 11). Any difference between the gaps with lightweight concrete or another glulam timber beam was not observed.

[3.1] = [1.14]

[3.2] = [1.15]

[3.3] Kordina, K. and Meyer-Ottens, C. 1983. Holz-Brandschutz-Handbuch (in German, title translates to - Timber - fire protection handbook). German Association for Timber Research. Munich.

[3.4] Kordina, K. and Meyer-Ottens, C. 1982. Brandprufungen an Holz und Holzverbindungen (in German, title translates to - Fire testing of timber and timber joints). Fraunhofer Association, Information Centre RAUM and BAU, Research Report T934. Stuttgart.

4. THE EFFECTS OF METAL DETAILS ON THE CHARRING RATE

Metal details in contact with timber surfaces can influence the penetration rate of the pyrolysis zone in several different ways:

(1) The heat conduction conditions at the contact surface are changed.

(2) If the stresses at the contact surface exceed the compressive strength of charcoal, compression of the charred layer can occur which reduces its insulating efficiency.

(3) In situations where the surface stress is non-uniform or the contact area is small in relation to the thickness of the charred zone, the charcoal, instead of being compressed, can be crushed when the compressive strength is exceeded; thus its insulating function is partially or completely destroyed.

(4) Steel fixing elements, e.g., nails or screws, can transmit heat energy into the material itself. Depending on their locations, they can increase the penetration rate or, as an exception, conduct heat away from the pyrolysis zone thus delay the penetration locally.

(5) Steel surface is air-tight. This makes it difficult for the movement of gases away from the pyrolysis zone.

4.1 Plates, Gusset Plates and Nail-on Plates

Aarnio and Kallioniemi [4.1], [4.2], [4.3] have performed some fire tests with unloaded flat plates 8 x 75 and 8 x 125 mm (Figure 4.1) and shown that after 15 minutes of testing in accordance with the standard fire conditions of ISO 834, the charred depth under the two types of plate was 6 mm compared with an average of 9 mm for the beam. After 60 minutes of fire, no difference in charring characteristics was found. The latter results were also obtained by Lihavainen [4.4] in a similar test. It was noted that the flat plates were fixed in position, thus contact was lost soon after the beginning of the test. It should also be borne in mind what Aarnio has demonstrated about the effects of gaps (Chapter 3). Hence the results after 60 minutes of fire test seem reasonable. The temperature in the flat plates recorded during the test and the corresponding results from Lihavainen's investigation are shown in Figure 4.2.

Aarnio has also shown the results of 60 minutes of fire test according to ISO 834 on an uninsulated steel plate with dimensions 185 x 200 x 8 being pressed on to the top of a glulam timber beam 185 mm wide. The compressive force was 15 kN which corresponded to a compressive stress of 0.4 MPa at the beginning of the test. This figure was well above the compressive strength of charcoal (0.3 MPa) which was determined in the same investigation. During the test, the compressive stress was gradually increased in the same manner as the width of the beam is reduced. Figure 4.3 shows the temperature of the furnace and that at the contact surface between the steel plate and the glulam timber, as well as the displacement of the steel plate during the test.

The temperature of the steel plate exceeded 300°C after 13 minutes. During the first 13 minutes, the rate of displacement was on average 0.5 mm/minute and thereafter it increased quickly to about 1.3 mm/minute which remained constant for the last 40 minutes of testing.

Ahlen and Mansson [4.5] have performed fire tests of two types of beam joints, one with bolted plates 10 mm thick as shown in alternative 1; and the other with fitted sleeve of 2 mm thick cold-formed steel plate and nailed as shown in alternative 2 of Figure 4.4. The temperature at the contact surfaces between the glulam timber and steel plates and fitted sleeve respectively are shown in Figure 4.5.

After only 5-10 minutes of testing, the temperature of 300°C was reached. The same average charred rate (0.7 mm/minute) were obtained for the steel plate bearing on the side surface, i.e., under a steel plate which did not transfer loads to the timber and a timber surface which was directly exposed to the fire. For the contact surfaces between the fitted sleeve and the glulam timber where load was transferred through bearing, the average charred rate measured was linearly related to the bearing stress; as shown in Figure 4.6. The same figure also includes the relationship between the charred rate and the proof stress of the bolted joint. It can be noted that Aarnio's result of 1.3 mm/minute at σ = 0.4 MPa fits closely the relationship obtained.

Rimstad [4.6] [4.7] and Bakke [4.8] have also performed 40 minutes of fire test according to ISO 834 on unloaded steel gusset plates which had been fitted onto 90 mm or 190 mm wide glulam timber beams with through bolts or wooden bolts of different sizes. Both uninsulated plates and plates insulated by different means were investigated (see later in Chapter 7). The aim of the tests was to study the temperature development in the metal parts; especially the time to reach a temperature of 300°C. The thermocouples were placed at the contact surface between the fittings and the glulam timber beam. Figure 4.7 shows the temperature development in various uninsulated and insulated plates. It is self-evident that the temperature of 300°C for uninsulated plates is reached after 10-15 minutes. However, the depth of the charred zone was not presented.

Hviid and Olesen [4.9] presented results of fire tests on timber joints with nailed plates under tension as shown in Figure 4.8. Both uninsulated and insulated joints were investigated. Besides the deformations of the joints, the temperatures were also measured at the nailed plate and in the middle of the timber itself. The temperature results for the uninsulated joint are shown in Figure 4.9. It is clear that the temperature in the nailed plate exceeded 300°C after about 7.5 minutes. Again, there was no measurement of the charred depth.

4.2 Bolts with or without Washers

Hviid [4.10] presented results from a preliminary study of the project "Penetration rate in wood loaded by steel dowels at elevated temperatures". This investigation aimed at determining the penetration rate in timber loaded by heated steel dowels as a function of:

(1) the steel temperature
(2) the stress level (embedding stress)
(3) the force-fibre angle
(4) geometrical dimensions

Instead of the conventional full scale fire tests, the tests were carried out with electrically heated steel dowels which were pressed against the timber specimen as shown in Figure 4.10. During these tests, the temperature and the embedding stress were maintained constant. The variation of various parameters included:

Steel temperature	340, 380, 420, 460, (500), (540) °C
Stress level	3, 6, 9, (12) MPa
Force-fibre angle	0°, 90°

Dowel diameter	12 mm, 25mm
Timber thickness	25mm

The figures given in parenthesis were not investigated for the force-fibre angle of 0°.

Hviid summarised the results as follows:

(1) The penetration rate increases by increasing steel temperature

(2) The penetration rate increases by increasing embedding stress

(3) At force-fibre angle of 90°C, there is interaction between the influence of the steel temperature and the embedding stress.

(4) At the higher steel temperatures and embedding stresses, the penetration rate is higher at force-fibre angle of 0° than at force-fibre angle of 90°.

(5) At the lower steel temperatures and embedding stresses, no influence of the force-fibre angle on the penetration rate can be demonstrated.

Hviid also pointed out that when the embedding stress is high, the penetration rate was constant throughout the test. If the stress is low, the rate was decreasing. The difference was explained as follows: when the embedding stress is low, a layer of charcoal can be built up in front of the dowel and the heat from the dowel must pass through this insulating layer which increased in thickness all the time. This reduced the temperature in the charred zone which subsequently reduced the penetration rate. When the embedding stress is high, the charcoal was crushed by the dowel thus no insulating layer was built up. Figures 4.11 and 4.12 show the results of 144 tests. The effects of different dowel diameter were not taken into account.

The final report for this ambitiously conceived investigation has not been published; probably because the work was interrupted.

In relation to this, it is worth mentioning that a rather comprehensive investigation has been performed using hot wires for cutting timber as an alternative to conventional sawing [4.11]. In this investigation, the wires used have diameters of less than 9 mm.

The results showed that at about 250-300°C, the wire began to cut, but the cutting was very slow and required a high bearing stress. The best result was obtained with a 0.35 mm Wolfram-wire together with a backward and forward sawing motion. When the wire was pressed into the track with a force of 28 N, a 25 x 25 mm timber section was cut in 9 seconds, i.e., the penetration rate was 167 mm/minute and the stress was about 3.2 MPa.

Leicester and Seath [4.12] have fire tested different kinds of timber joints, i.e., toothed plate connector joints, split ring joints, bolted joints and nailed joints, see Figure 4.13. Two different types of fire were studied and the results are given in Figure 4.14. Based on this investigation, the Australian Standards recommend 20 minutes as the fire resistance of unprotected bolted joints, see Chapter 10.

Ahlen and Mansson [4.5] investigated, among other subjects, beam joints with 10 mm thick steel plates and 25 mm diameter bolts, as shown in Figure 4.4, at two different load levels. They found that for a 30 minutes standard fire, a relationship existed between the calculated proof stress and the measured penetration rate, see Figure 4.6. The angle between the proof stress and the fibre direction in the most heavily loaded bolt was calculated to be 50°. For the proof stresses of $\sigma = 3$ MPa and $\sigma = 5$ MPa, the penetration rate was $v = 1.5$ mm/minute and $v = 2.1$ mm/minute respectively.

As a comparison, it is noted that Hviid [4.10] used 25 mm diameter bolts and obtained penetration rates of 1 mm/minute and 15 mm/minute with embedded stresses of 3 MPa (average over 14 minutes) and 6 MPa respectively! These results were obtained for embedding stresses parallel to the fibre direction and a steel temperature of 460°C.

The temperatures of the bolts 10 mm from the underside of the plate were recorded by Ahlen and Mansson and given in Figure 4.15.

Kordina and Meyer-Ottens [4.14] indicated 34-36 minutes of fire resistance for the beam-to-column joint in Figure 8.11, line 6.1, where the whole reaction is taken up by an inserted 10 mm thick steel plate and an uninsulated steel section in accordance with DIN 4102 [4.15] in class F 30-B. In the same joint detail, there was also an uninsulated dowel fitting with Bulldog type washers. If the load is reduced to 0.9 F_a, the edge distance e_3 can be reduced to 30 mm for the same fire resistance. If the load is further reduced to 0.3 F_a, e_3 can be reduced to 20 mm accordingly.

For bolted connections, e.g., those with external gusset plates and uninsulated bolts with Bulldog type washers or similar, as shown in Figure 4.16, the same source [4.14] indicates their fire resistance is 15 to 16 minutes. If these bolts and washers are protected with the coverings secured by at least four nails of sufficient anchorage length, their fire resistance increases to 30 minutes. This can be compared with what has been described in Chapter 3 on the effects of gaps and openings in the joints. Figure 4.17 shows the deformations of this type of fitting during standard fire tests (investigations 1 and 2). When there are at least six nails used per bolt or washer, the joint in Figure 4.16 is classified as F 30-B according to DIN 4102 [4.15].

For similar types of connection, but with an inserted plate of sawn timber or plywood and an uninsulated dowel fitting as shown in Figure 4.18, Kordina and Meyer-Ottens [4.14], [4.16] indicated, on the basis of testing, fire resistances of 32 and 30 minutes for timber and plywood plates respectively at the maximum permissible load. At 50% load ratio, the corresponding figures were 41 and 32 minutes respectively. The results can also be assumed to be valid for ordinary bolt fittings if the boundary conditions are similar, e.g., edge distance. The joint is classified as F 30-B [4.14], it is also stated that the load ratio must be 50% maximum for plywood plates.

Aarnio [4.1], [4.2] has investigated the temperature developments in bolts under 35 minutes of standard fire. Four different bolt sizes were studied, i.e., 12, 16, 20 and 24 mm diameter, all fastened to 185 mm wide glulam timber beams. It is not clear whether washers had been used, nor their sizes. All the bolts were unloaded during the tests. Figure 4.19 presents the furnace temperature, and the temperature at the head and middle of the various bolts. It is evident that the heating process is substantially quicker for the two smaller bolts than the bigger ones; which Aarnio explained was related to the greater mass of the bigger bolts. When the beams were sawn after the test, all the timber around the bolt holes was more or less completely charred. The extent of the charred zone was not presented.

Rimstad [4.6], [4.7] and Bakke [4.8] have also measured the temperature of a 25 mm diameter bolt in 90 mm wide glulam timber during 40 minutes of fire test. The measurement points were situated at the contact surface between the steel and the timber. From the results, the temperature at the head of the bolt after 30 minutes was 600°C and 750°C for the first and second tests respectively, which correlates well with Aarnio's result of 750-800°C for 12-24 mm bolts.

4.3 Nails

The fire resistance of nailed gusset joints as shown in Figure 4.10 [Translator's note: Figure 4.8] has been investigated by Haviid and Olesen [4.9]. Both uninsulated and insulated joints, according to Figure 7.10, have been studied. The nailed joints consisted of 4x15 nails 40/60 with a total load of 16.3 kN, i.e., 540 N/nail. It is likely that the timber used for insulation may contribute, at least initially, to the transfer of the applied forces. The insulation was nailed with 4x7 nails 40/60. When these worked together, the load for the insulated joints became only 370 N/nail at the beginning of the test and during the process of the test, it increased to 540 N/nail. The temperature in the gussets as well as the deformation (lengthwise, and excluding the initial deformations) were measured and the results shown in Figure 7.11. The fire resistance for uninsulated joints was indicated as being 12 minutes and for the insulated joints 36 and 37 minutes respectively, depending on the type of insulation used. It is not clear from the study to what extent the plywood used, which was impregnated with fire retardant, had charred during the test.

Leicester and Seath [4.12] also investigated the fire resistance of some timber joints with nailed splice plates as shown in Figure 4.13. Two different types of fire exposure were studied and the results are given in Figure 4.14.

Similar types of nailed joints with dimensions shown in Figure 4.20 were fire tested by Aarnio et al [4.13]. The nailed joint consisted of 4x42 threaded nails (?) 60x25 [Translator's note: 4 x 4.2 (?); 60 x 2.5] and the load applied was 16 kN, i.e., 190 N/nail. The failure load at room temperature under short term loads (time to failure = 12 minutes) was found to be 67 kN in a special test, i.e., 800 N/nail. The test fire condition was in accordance with ISO 834. The temperature at the joint surfaces was recorded but not the deformation of the joint. The failure criteria was not defined and the given fire resistances were only approximate. The mean value for the three tests was 17.5 minutes. The total number of fire tested joints was only 9 because each time the test was carried out with three joints connected in series, and the test was terminated after the weakest joint collapsed. Aarnio pointed out that the number of effective nails was slowly decreasing as the timber charred, which was why the load in the remaining nails gradually increased. Furthermore, it was noted that in those joints which were pulled to failure, the nail heads actually pulled through the gussets. A probable explanation was that at the time of failure, the temperature at the tip of the nail was relatively low thus the anchorage was still effective. But the high temperature at the nail heads caused the surrounding timber to char thus offering very little resistance to pull through.

The temperature at the adjoining surface was measured at two locations in each joint, i.e., 30 mm and 60 mm respectively from the underside of the beam. At the upper measurement points, the temperature was consistent but remarkably low, i.e., the mean value for six points was 122°C after 15 minutes of fire (standard deviation 38°C) while considerably higher values were recorded at the lower measurement points, i.e., the mean value was 500°C for six points (standard deviation 180°C); probably because of the fact that these points were closer to the nails. According to Figure 4.20, the theoretical distance to the nearest row of nails was 9 mm for the upper measurement points and 3 mm for the lower measurement points.

Aarnio has also, in early investigations [4.1], [4.2], studied the charring characteristics around unloaded nails. Several different types of nail were investigated : 60x25, 75x28, 100x34, 125x42 and 150x51 [Translator's note: 60 x 2.5, 75 x 2.8, 100 x 3.4, 125 x 4.2 and 150 x 5.1] in groups of four nailed into the side of the timber with minimum spacings according to the Finnish Regulations, i.e., 5 d and 10 d in directions perpendicular and parallel to the grain respectively. After 35 minutes of the standard fire, the samples were sawn through. It was found that the charred zone had reached 20-30 mm deeper in the immediate vicinity of the nails with the large nails resulting in somewhat more penetration. Between the nails, the charred zone had penetrated 5-10 mm deeper than in the sections without nails; with the small nails giving a greater increase of penetration depth than the large nails; as illustrated in Figure 4.21.

Aarnio and Kallioniemi [4.2] have also presented results from similar tests using 5.1 mm nails of different lengths (115-145 mm) and 70 minutes of exposure to the standard fire exposure. On the side from which the nails were inserted, the penetration of the charred zone between the nails increased by 10-20 mm. But locally, by the side of the nails, the penetration reached about 60 mm more which was independent of the length of the nails. On the opposite side, it was noted that the depth of the charred zone had decreased locally along the tip of the longest nails, which after 70 minutes of fire, almost reached through the uncharred part of the beam. The apparent reason for this was that the nails in this case had conducted away the heat from the charred zone on the "backside".

Kordina and Meyer-Ottens [4.14] have performed tests on joints with external nail-on gussets of sawn timber or glulam timber as shown in Figure 4.22. For unprotected nailed joints, the fire resistance was given as 22 minutes which can be compared with the 17.5 minutes given by Aarnio. With the provision of 24 mm and 40 mm protection plates, the fire resistance increased to 48 minutes and 68 minutes respectively. The deformations of the joints are given in Figure 4.23. According to DIN 4102 [4.15], the timber protection thicknesses of 24 mm and 40 mm are classified as class F 30-B and F 60-B respectively.

For the same type of joint with splice-in plates of sawn timber (Figure 4.24) or 2 mm thick steel (Figure 4.25) and unprotected nailed joints, Kordina and Meyer-Ottens [4.14], [4.16] indicated that the fire resistance was 27 minutes for the timber splice and 47 minutes for the steel splice, with the applied load equal to the maximum permissible. Joints with timber splices were also tested at lower temperatures whereby the fire resistance was increased to 36 minutes at 85% and 42 minutes at 66%. These were classified [4.14] as class F 30-B where the capacity may be, at the most 85%, for a timber splice.

In order to establish the length of nails or screws required for the fastening of fire resistant ceilings consisting of gypsum plasterboards, Anderberg [4.17] examined the withdrawal resistance of ten different types of nail and screw after 15, 30 and 60 minutes of the standard fire. The nails and screws were of a type which are normally recommended by the gypsum plasterboard manufacturers for fastening their products. The withdrawal resistances before and after the fire together with the maximum temperature at the tip of the nail or screw were recorded.

The results show that the withdrawal resistance after the fire test, besides being related to the type of nail or screw, also depends on the anchorage length and the maximum temperature at the tip of the nail or screw.

4.4 Toothed Plates

The fire resistance of toothed plate joints with bolts have been investigated by Aarnio et al [413]. The plates were of type NLOY 100x200x1.5 mm^3 - judging from the photographs there were about 180 teeth on the plate, 12 mm long and 2.5 mm wide. The bolts were made from construction timber in durability class T 30 with dimensions of 45x120 mm^2. Both completely uninsulated joints and joints insulated on three sides with 100 mm thick mineral wool, with density of 150 kg/m^3, were tested.

The temperature in the toothed plates was recorded but not the deformation of the joint. Failure load at room temperature (time for breaking 8-9 minutes) was measured to be 61 kN in a special test. The tension in the joint, i.e., 16 kN, was kept constant during the fire tests. The fire resistances of the uninsulated joint and the insulated joint were found to be 7.5 minutes and 42 minutes respectively. The results were the mean values for three and four tests respectively, i.e., a total of nine and twelve joints respectively were tested in the following arrangement : three joints were connected in series in a test. Thus the measured value constituted the fire resistance of the weakest joint in each group. Failure took place in every case in the anchorage caused by the rapid charring of the timber around the teeth of the plate. It should be noted that for the tension tests performed at room temperature, failure in both cases was by rupture of the plate.

The temperature in the joints was measured for both insulated and uninsulated joints. In five of the nine uninsulated joints, the temperature after 6 minutes was already higher than 300°C (mean 314°C, standard deviation 60°C) while only one of the nine insulated joints had exceeded 300°C after 39 minutes (mean 242°C, standard deviation 53°C).

Leicester and Seath [4.12] have also investigated similar toothed plate joints, see Figure 4.13. Two different fire situations were studied. The results are given in Figure 4.14. On the basis of these tests, Australian Standards recommend 5 minutes as a guideline for the fire resistance of unprotected toothed plate joints (for details refer to Chapter 10).

Fire tests of toothed plate joints have also, according to our information, been carried out by the Truss Plate Institute, Madison, USA, but the results are not available.

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[4.2] = [1.15]

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[4.11] Cutting by heated wires.

[4.12] Leicester, R.H., Seath, C. and Pham, L. 1979. The fire resistance of metal connectors. Proceedings Nineteenth Forest Products Research Conference, Melbourne.

[4.13] Aarnio, M., Holm, C. and Kallioniemi, P. 1980. Sahatavaran vetoliitosten palonkestavyys (Dragna virkesskarvars brandsakerhet) (in Finnish, title translates to - Fire safety of timber joints) State Technical Research Centre, Fire Research Laboratory, Report No. 22. Esbo.

[4.14] = [3.3]

[4.15] German Standards Institution. 1981. Fire behaviour of building materials and building components. DIN 4102, Part 4. Berlin.

[4.16] = [3.4]

[4.17] Anderberg, Y. 1980. Undersokning angaende skruvs/spiks fasthallande effekt vid brandpaverkan (in Finnish, title translates to - Investigation into the holding capacities of nails and screws during fire). Royal Institute of Technology, Institute for Building Research, Internal Report. Lund.

5 THE EFFECTS OF CONCRETE DETAILS ON THE CHARRING RATE

The joints between concrete and timber elements normally occur at junctions, i.e., at the top of a concrete column as in Figure 5.1a or at supports as in Figure 5.1b. Also in exceptional cases, in concrete roof construction, or more commonly, lightweight concrete slab on glulam timber beams, see Figure 5.1a-b, or glulam timber column placed directly in concrete, Figure 5.1c.

Aarnio [5.1], [5.2] has fire tested beams in Figures 5.1a and b as well as studied the effects of fire on a glulam timber beam with the top of the beam protected by lightweight concrete. Figure 5.2 shows vertical sections across and along the beam in Figure 5.1a after 60 minutes of standard fire.

Figure 5.3 shows the horizontal and vertical sections along the beam in Figure 5.1b after the same test period. The loads applied during the tests were 0.04 MPa and 0.06 MPa respectively, which corresponded to the concrete self-weight. The temperatures at the surfaces were measured together with the temperature in the gap between the side of the beam and the concrete in b.

[5.1] = [1.14]

[5.2] = [1.15]

6. GLUED JOINTS, FINGER JOINTS

There is very little published information on the fire performance of glued joints. However, experience from fire testing of glulam timber beams indicates that a well-manufactured glued joint can be more durable than the surrounding timber even in a fire situation. Tenning [6.1] has investigated both casein and resorcinol glued beams and did not observe any melting nor other negative performance.

Imaizumi [6.2] reported that in tests of carbamid glued joints of Japanese spruce (Picea jezoensis Carr), the charred zone was somewhat deeper at the glued joint that in the middle of the timber between laminations. But the glued joints at the uncharred section of the beam were intact.

Nyman [6.3] and Kallioniemi [6.4] have shown, on the basis of Finnish investigations, that the proportion of fracture during shear testing of glued joints increased with increase in temperature (between +20 and 200°C) and have drawn the conclusion that the timber material and not the glue was the weak link in the chain. The glue used was conventional phenol-resorcinol glue.

Malhotra and Rogowski [6.5] studied the effects of different types of glue on the fire resistance of glulam timber columns and found that the columns which had been glued with phenol glue had the best fire resistance whereas casein glued columns gave 20% lower results, see table below.

Type of glue	Relative fire resistance for glulam timber columns	Relative rate of charring: Parallel to the glue line	Relative rate of charring: At an angle to the glue line
phenol	100	100	100
resorcinol	94	105	97
carbamid	92	111	118
casein	82	120	119

Rogowski [6.6] found similar results of the influence of the glue on the charring rate, see table.

Aarnio et al [6.7] investigated the fire resistance of finger joints of T30-timber with cross-sectional dimensions of 22 x 120 mm² and 45 x 120 mm² under loads of 16 kN and 32 kN respectively. In both cases, the stress at the beginning of the test corresponded to 6 MPa. Failure loads at room temperature were measured to be 101 kN and 206 kN in special tests which corresponded to stresses of 40 MPa and 38 MPa respectively. For 22 mm thick timber, a fire resistance of 9 minutes was obtained while that for 45 mm thick timber was 16.5 minutes. The results constituted the mean values of four observations, each corresponding to the fire resistance of the weakest of three joints connected in series. Altogether, twelve joints were tested in each dimension but only four were pulled to failure. Of the total eight failures, seven were in the finger joint itself at the lower part of the joint. Furthermore, in most cases, the failure was a combination of the finger joint and a knot or a timber defect in the vicinity, see Figure 6.1. Careful measurements at the end of the test showed that the mean stress at failure was about 15 MPa with very little variation.

The temperatures at the centre of unloaded timber beams with dimensions 22 x 120 mm^2 and 45 x 120 mm^2 were also measured. According to these, the temperatures at the centre of the cross-section must have been 150°C and 100°C respectively at failure.

Nielsen and Olesen have also studied the load capacity of finger joints at elevated temperatures. In a pilot study [6.8], they fire tested a number of jointed and unjointed glulam timber members in strength class T40 with a cross section of 33 x 139 mm^2. The load applied in the tests corresponded to a stress of 12.5 MPa for unjointed timber and 11.1 MPa for jointed timber. The results indicated that the weakening, which the finger joint had contained within itself, significantly increased during the fire. Five unjointed samples achieved on average eight minutes fire resistance (Mv = 488 s, s = 65 s) and the four jointed samples on average six minutes fire resistance (Mv = 340 s, s = 19 s). In a later investigation, Nielsen and Olesen [6.9] examined finger joints (which matched unjointed timber in strength class T30 and with a cross-section of 33 x 90 mm^2) by heating them to a constant temperature and then testing until failure. The heat was applied with the help of electrical elements and steel plates in direct contact. Four temperature levels were investigated, from 20 to 230°C. The results are shown in Figure 6.2 and can be summarised as follows:

(1) The strength is strongly dependent on the temperature both for unjointed and finger jointed timber.

(2) At 90°C, the strength of the timber without joints was significantly higher than that with finger joints.

(3) When the temperature was between 160 and 230°C, the finger joints have no effect on the strength.

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[6.6] Rogowski, B 1970. Charring of timber in fire tests. Proceedings Symposium No 3: Fire and Structural Use of Timber in Buildings, Fire Research Station, Boreham Wood, Herts, 25 October 1967. Her Majesty's Stationery Office. London.

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[6.9] Nielsen, P and Olesen, B. 1982. Tensile strength of finger joints at elevated temperatures. Aalborg University Centre (AUC), Institute of Building Technology, Report No 8205. Aalborg.

7. FIRE PROTECTION OR INSULATION OF METAL DETAILS

Experience to date shows conclusively that only in exceptional cases, can joint details using exposed metal parts reach higher fire resistances than 15-20 minutes, i.e., as a rule far below the capability of the adjoining timber components. Hence the appropriate action would be to protect the metal parts against direct fire damage in order to delay the temperature build-up in these parts. Some of the insulation methods used are similar to those in large steel structures, e.g., fire retardant paint, fire resistant plaster or coatings; and others may include the use of incombustible board, timber board or boards of wood-based materials. For nails and bolts, a common method is to bury them into the timber and cover them with wooden plugs. Sections 7.1 to 7.4 describe these methods [7.1].

7.1 Fire Resistant Coatings

One of the salient features of fire resistant coatings is that when they are heated to 100-150°C, they swell significantly and form an insulating foam. Approval List B of the State Planning Commission [7.2] lists three products of this kind. The total required coating thickness is normally 1-2 mm and this can easily be achieved by two to three layers of coating. As a rule, some form of top coat is also required. These coatings cannot be used in an external environment.

A lot of experience with these coatings has been gained and various tests well documented. However, they are mainly associated with large steel structures. For small units with a mass less than 50 kg, these results may not be applicable unless substantiated by further investigations.

Rimstad [7.3] and Bakke [7.4] have shown temperature results from 40 minutes of fire testing of 8 and 16 mm thick unloaded steel plates with masses from 1.5 to 34 kg screwed on to the side of 90 mm wide glulam timber beams and coated with three layers of "Unitherm" fire resistant coating on the free surface. As a comparison, they also measured the temperature development in unprotected, but in all other aspects similar, plates. From the results, which are partly included in Figure 4.7, it is clear that the fire resistant coating had no effect until the temperature reached beyond 150-200°C (or above) and the temperature delay is about 10 minutes at temperatures of 300-500°C. Rimstad used the attainment of 300°C at the surface between the steel and the timber as his failure criterion and gave the time to failure as 23 minutes for the unprotected case (s = 6.9 minutes, 17 measurement points) and 28 minutes for protected case (s = 5.5 minutes, 9 measurement points). The time for the unprotected case seems incorrect and should probably be 14 minutes.

Kordina and Meyer-Ottens [7.5], [7.6] have tested round bars of different types used in steel structures which were treated with a fire resistant coating called "Pyrotect S 30". On the basis of these tests, they classified steel bars of diameter 27 mm or bigger treated with "Pyrotect S 30" as class F 30. All exposed connections should also be protected in a similar manner. Allowable proof stress may be used up to 30%. A summary of these results is given in Figure 7.1.

7.2 Fire Resistant Plaster

"Fire resistant plaster is the name given to fire resistant materials where vermiculite or perlite make up the bulk. As a bonding agent, one usually employs cement or gypsum. Vermiculite is an expanded mica material and perlite an expanded lava. Plaster can be sprayed on or applied by hand onto the steel construction. The thickness normally varies between 10 to 40 mm depending on the type of plaster used and the insulation requirements. For certain types of plaster, it is recommended that the maximum thickness for each layer should be 10-15 mm before the next layer is applied. This is to permit the previous layer to dry out, and the plaster must be applied at temperatures higher than 0°C. Certain plasters can be applied directly onto the steel surface whereas others may require a steel mesh in order to achieve sufficient adhesion. The surface can also be painted. When certain plasters are used, the surface can be quite rough and it is recommended that putty can be used to even the surface before painting.

The density γ_i normally varies between 300 to 800 kg/m^3 depending on the type of plaster. Plaster with a lower density are relatively soft and this must be taken into account when they are used in structures with a high risk of mechanical damage. An increase in the mechanical durability can be achieved through application of fabric materials on the surface. Sometimes, some form of corner protection is also necessary. If mechanical durability is desirable, the surfaces can also be covered with a hard surface plaster. For plaster of higher density, the surface is often relatively hard." [7.1]. The Planning Commission Approval List B [7.2] only shows one product of this kind, i.e., "Jimotherm" which has a density of 800 kg/m^3.

The conductivity for "Jimotherm" as a function of the temperature in the plaster is shown in Figure 7.2.

Rimstad [7.3] and Bakke [7.4] have also measured the temperature of steel plates protected with a 20 mm fire resistant plaster with the test arrangements similar to the one described in section 7.1. The results are given in Figure 4.7 and show that the plaster, in contrast to the fire resistant coatings, is effective at the early stage of the fire, giving a temperature delay of about 15 minutes at temperatures of about 300-500°C. Rimstad indicated, with the critical temperature being 300°C, that the fire resistance is on average 33 minutes for plates protected with 20 mm "Pyrocrete 102" (s = 3.4 minutes, 9 measurement points).

7.3 Sprayed Mineral Wool

"Sprayed mineral wool consists of mineral wool fibres mixed with concrete or cement as a bonding agent. This mixture is sprayed together with water in a fine particle form to the desirable thickness directly onto the surface which is to be insulated. The thickness of the insulation is normally between 10 to 30 mm depending on the insulation capacity required. The spraying must be done above freezing point. Before spraying, any oily material or dirt must be removed from the surface. Minor rusting, however, does not impair adhesion. The surface should also be suitable for spraying. The density of the spray material varies depending on the mix or manufacturer and can vary between 250 to 370 kg/m^3. The insulation is relatively soft and this has to be taken into account in applications where there is a risk of mechanical damage. Improved mechanical strength can be achieved with the inclusion of glass fibre materials onto the surface." [7.1]. Planning Commission Approval List B [7.2] only shows one type of mineral wool spray, ie, "Sprayed Limpet" mineral fibre GP grade with a density of 250 kg/m^3. The conductivity and heat absorption characteristics as a function of the temperature of a similar product with the same density are given in Figure 7.3a and 7.3b.

7.4 **Boards Of Incombustible Material**

Boards for fire protection can be made from mineral wool, vermiculite or gypsum. Planning Commission Approval List B [7.2] only deals with mineral wool boards and gypsum plasterboards.

Mineral wool boards should have a minimum density of 150 kg/m^3. Normally they have a thickness of between 30 to 70 mm. For steel surfaces, these are fastened with nails and special discs. In the surrounding timbers, nails and metal discs can be used. In cases where mechanical damage is possible, the mineral wool boards must be provided with special surface protections.

Mineral wool boards in circular shape have been used to insulate tie rods. The conductivity and heat absorption characteristics against the mineral wool board temperature are given in Figure 7.4a and 7.4b.

Aarnio and Kallioniemi [7.7] [7.8] referred to an investigation carried out by Lihavainen [7.9] which compared the fire resistance of an unloaded beam-to-column joint, as illustrated in Figure 7.5, with and without a 30 mm thick mineral wool insulation (γ = 150 kg/m^3). The temperature development at the gusset-timber contact surface was measured during the fire test and the results given in Figure 7.6.

Aarnio and Kallioniemi have also studied the effects of mineral wool insulation on the temperature development in 75 and 125 mm wide flat plates and the charring of the timber around these plates. The dimensions of their test samples are given in Figure 7.7. The mineral wool is 30 or 50 mm thick with a density of 150 kg/m^3. They were fastened with nails and glued to the timber surface. No forces other than self-weight is applied between the plate and the timber. The results of the temperature measurements are given in Figure 7.6. Figure 7.8 shows the extent of the charred zone after 60 minutes of fire.

Gypsum plasterboards, with a density of 800 kg/m^3, have been used for fire protection of steel structures and they are also suitable for the protection of joints in glulam timber construction. Normally, 13 mm thick boards are used in one, two or three layers depending on the insulation requirements. They can be fastened onto steel with screws and onto timber using nails or special screws. Since metal joints often have irregular and complicated shapes with protruding screw heads etc., the protections are usually in box form with mineral wool infill.

The conductivity and heat absorption characteristics of gypsum plasterboards as a function of the insulation temperature are given in Figures 7.9a and 7.9b.

Asbestos-cement boards are no longer permitted in Sweden and a substitute with similar fire resistant characteristics, i.e., "Minerit Byggskiva", have been classified by the Planning Commission as inflammable protective boards with a non-flame spread surface (class 1).

Aarnio and Kallioniemi [7.7], [7.8] have also investigated the effect of placing a 5.5 mm asbestos-cement board between the timber and the metal detail; in this case it was a 75 mm by 125 mm flat plate with no force applied to the joint. When compared with an uninsulated plate, it was found that the asbestos-cement board had reduced the charred depth by approximately 30%. The testing time was 20 minutes.

7.5 Boards or Plugs of Timber or Wood-Based Material

The fire protection of metal details can also be achieved by using timber or wood-based materials, or by burying the nail or bolt head into the timber construction and plugging the holes with wooden plugs; which can also be glued in. The detail can also be designed such that the metal parts are not exposed to the fire but are protected by the adjoining timber elements. More discussions on this topic are presented in Chapter 8.

Hviid and Olesen [7.10] have performed fire tests of nailed gusset joints in timber construction as shown in Figure 4.7. [Translator's note: Figure 4.8] Unprotected joints (see section 4.1) and protected joints as shown in Figure 7.10 were tested. In this case, the temperature of the gusset as well as the elongation of the joint under a constant load of 16.3 kN, was recorded. The results are shown in Figure 7.11. Tests C01 - C03 were terminated after 30 minutes whereas tests C04 and C05 were terminated after the joints ruptured at 36 and 37 minutes respectively. The failure criterion used was not indicated. For test samples C01 - C03, the load was held constant (16.3 kN) for approximately 25 hours after the fire test; thereafter the load was increased by 50% to 25 kN and was then held constant for another 5 minutes. For samples C02 and C03, the load was increased further to 30 kN without failure. For sample C01, the load was increased to 81 kN where fracture occurred in the timber at the side of the joint. The total elongation of the joint, measured from the beginning of the fire test, was about 13 mm. In Table 7.5, the deformations of the joints C01 - C03 during and after the fire tests are presented.

For uninsulated joints, i.e., sample C06, the load capacity was exhausted after 12 minutes. A direct comparison of results for insulated joints was complicated by the fact that the plywood insulations were fastened with nails which, at least at the beginning of the fire test, must be assumed to have been involved in the heat transfer. From the data one can however draw the conclusion that the various protections tested delay the increase of temperature and deformation in the joint by approximately 25 minutes and prolong the fire resistance by approximately 20 minutes.

Aarnio and Kallioniemi have, in earlier investigations [7.7], [7.8], studied the protection performance of boxes made of glulam timber or particle board. The boxes were put over unloaded steel gussets which were nailed to the side of the timber beam. The temperature in the steel gusset was measured during a standard fire. Gussets of two different sizes and three types of boxes were tested, see Figure 7.12.

- Glulam timber box, joined and fastened to the beam with nails.
- Glulam timber box, joined and fastened to the beam with glue and nails.
- Particle board boxes, joined and fastened to the beam with glue and nails, and any internal void filled with mineral wool.

The glulam timber boxes were made from two different thicknesses of timber, i.e., 40 mm and 70 mm. In order to investigate the effects of leaks or gaps in the boxes, one box of each timber thickness was prepared with a 1 mm gap between the top and the side protections.

The temperature results can be summarised as follows:

- Glulam timber boxes which were glued to the side of the timber effectively delay the temperature rise in the gusset. After 30 minutes of standard fire, the temperature had only increased 15°C in the box with 40 mm thick timber protection and not at all in the box where the protection was 70 mm thick.
- Even particle board boxes filled with mineral wool gave reasonably good protection. After 30 minutes of standard fire, the temperature had not exceeded 300°C and the particle board protection was completely charred; even so, the mineral wool was still in place.
- In cases where the boxes were nailed on only, gaps in the joints between the box and the side of the beam appeared relatively soon. Thereafter, convection through these gaps dominated the heat flow but there was no difference in the temperature development in boxes with different mineral wool thicknesses nor is there any effect of the 1 mm gap which was provided initially in one of the boxes.

Aarnio and Kallioniemi in the same investigation also studied the temperature development during a standard fire in a bolt which was embedded into the side of the timber with a 40 mm thick wooden plug glued into the hole. The temperature increase in the bolt, after 20 minutes of fire, was insignificant. The wooden plug had charred in the same way as the timber beam.

Kordina and Meyer-Ottens [7.5] have investigated the fire performance of joints with timber protections applied on the outside of bolted joints with internal split ring connectors where the bolt head was embedded into the timber and protected in the same way as described previously. In Figure 7.13, the test samples together with the fire resistance versus the thickness of the wooden plug are given. In DIN 4102 [7.11], the bolt joint protected in this manner is classified as F 30-B if the thickness of the wooden plug is 20 mm and as F 60-B if it is 40 mm.

The same reference [7.5] also shows the results of fire tested nailed gusset joints of sawn timber or glulam timber, as illustrated in Figure 4.20. For unprotected nailed joints, the fire resistance given was 22 minutes. With 24 mm and 40 mm thick protections, 48 and 68 minutes of fire resistance were given. In Figure 7.13, these results are also included in the fire resistance versus thickness diagram. The joint deformations are shown in Figure 4.21. The protections with timber coverings of thicknesses 24 mm and 40 mm are classified as class F 30-B and F 60-B respectively according to DIN 4102 [7.11]

Kordina and Meyer-Ottens [7.5] also indicated that the required thickness of nailed timber, plywood or particle board protections over a bolted joint (as shown in Figure 7.14) should be 20 mm for 30 minutes of fire resistance. However, the recommendation was based on speculation as no test results were given.

[7.1] = [1.1]

[7.2] Swedish Building Services. 1985. State Planning Commission, approval List B: approval for fire protections. Stockholm.

[7.3] = [4.6]

[7.4] = [4.8]

[7.5] = [3.3]

[7.6] = [3.4]

[7.7] = [1.14]

[7.8] = [1.15]

[7.9] = [4.4]

[7.10] = [4.9]

[7.11] = [4.15]

8. FIRE TESTED JOINT DETAILS

Only a limited amount of information on fire tested joint details is officially documented. The most comprehensive investigation was performed by Kordina and Meyer-Ottens and presented in [8.1] and [8.2]. Also, Aarnio and Kallioniemi [8.3], [8.4] as well as Ahlen and Mansson [8.5] have made significant contributions in this area.

8.1 Knee Joints of Glulam Timber Columns

Kordina and Meyer-Ottens [8.1] indicate that for column bases where the load is transferred through bearing, e.g., that shown in Figure 8.1a-c, are suitable even to satisfy a very high fire class requirement. If horizontal forces have to be resisted by external brackets as shown in Figure 8.1a, the authors consider that a fire class of F30 can be achieved without any protection for the steel. If horizontal forces are resisted by a concrete block as shown in Figure 8.1c, they estimate that a fire class of F60 or higher can be relied on.

According to the design given in Figure 8.2 where the transfer of load is through dowels or bolt fixings, the same source indicates a fire class of F30 if the minimum dimensions are satisfied. The conditions $a <= 100$ and $b \geq d/3$ are satisfied by the basic requirements for fire design according to DIN 4102: that the transfer of load during fire must be able to take place through bearing.

For a column base as shown in Figure 8.3, Kordina and Meyer-Ottens estimate that class F30 can be achieved by applying fire resistant coatings to the flanges of the steel section, while a higher class may require coatings of mineral wool, fire resistant plaster or similar materials. The bolts must be buried deeply and be protected by glue-in timber plugs or covering plates with thicknesses 20 mm and 40 mm for F30 and F60 respectively. The least edge distance e^1 is given as 60 mm and 90 mm respectively.

Wind bracings and their connections which are made of steel complicate this type of construction because even these must be fire-proofed to the same class as the whole construction. For the column base shown in Figure 8.4 under vertical load, the load is transferred by bearing through contact with the top plate of the I-section. Thus this detail, according to Kordina and Meyer-Ottens, does not require special protection for F30 to be achieved; provided that the gap dimension satisfies $50 \text{ mm} \leq a \leq 100$ mm and the dimension e_1 for the timber covering plate be $e_1 \geq 40$ mm. On the other hand, the anchorage of the bracing (e.g., nuts, tubes and sockets) must be protected, because these parts are considered to be particularly sensitive to temperature. In the examples shown, the authors recommended that the anchorage be covered with mineral wool or similar materials and the space between the flanges of the I-section be filled up with insulating materials.

8.2 Joints of Glulam Timber Beam to Timber, Concrete or Steel Columns

(a) Glulam timber columns with glue-in screws as shown in Figure 8.5 have been tested under 60 minutes of fire by Aarnio and Kallioniemi

[8.3], [8.4]. The temperatures in the screw and at the interface between the beam and the column were recorded. The results are given in Figure 8.6. The joint was not loaded. Aarnio and Kallioniemi found that, from a fire resistance point of view, the beam and the column have functioned as a unit and that no charring occurred at the interface. This finding was somewhat astonishing given the high temperatures measured at the interface. The joint between the screw and the timber was inspected after the test and found to be intact.

(b) Timber columns with gussets of glulam timber as shown in Figure 8.7 were also tested by Aarnio and Kallioniemi on the same occasion. The joints were also unloaded. The testing time was 60 minutes and the temperatures at the interface between the gusset and beam as well as at the bearing surface of the beam on the column were recorded and the results given in Figure 8.6. In Figure 8.8, the vertical and horizontal cross-sections of the sample after the test are illustrated. The interface between the undamaged part of the timber gusset and the beam was uncharred. Aarnio and Kallioniemi stressed that although the use of timber gussets may result in a larger undamaged cross-section than joints involving steel gussets, it is estimated that because of the problem of lateral stability and the poor performance of nailed joints, the fire resistance of timber gusset joints is similar to that with steel gussets. As a suitable improvement, they suggested that the nails be inserted more deeply or protected with a suitable plate.

For beams with depths less than 4 times their width, Kordina and Meyer-Ottens [8.1] indicated a fire resistance of F30 if the gusset is more than 25 mm thick and the width of the beam is more than 80 mm; and a fire resistance of F60 if the gusset is more than 40 mm thick and the width of the beam is more than 120 mm. For beams with depths greater than 4 times their width, the corresponding dimensions should be increased to 80 mm and 140 mm respectively due to the risk of instability.

(c) Glulam timber columns with side plates as shown in Figure 7.5 have been tested under 60 minutes of fire by Lihavainen [8.6]. The joints were not loaded during the test. Both uninsulated and insulated details have been studied. The insulation consisted of 30 mm thick mineral wool with a density of 150 kg/m^3 which was nailed and glued to the side of the glulam timber. Temperatures in the flat plates are given in Figure 7.6. Figure 8.9 shows the cross-sections after the test.

Similar tests have been carried out by Aarnio and Kallioniemi [8.3], [8.4]. The joints were also unloaded. Two different widths of plate were studied as well as the effect of placing a 5.5 mm thick asbestos-cement board between the plate and the timber. The joint with the asbestos-cement board was tested for 30 minutes while the other test was interrupted after 15 minutes. The temperatures in the contact surface between the plate and the asbestos-cement board and the glulam timber were recorded and the results are given in Figure 4.2. Results on the latter test shown a large variation thus it is difficult to draw any conclusion on the effect of the asbestos-cement board. On the other hand, it was found that the penetration rate of the charred zone under the plate, regardless of its width, was 30% lower when the asbestos-cement board was present. When there was only the plate, the penetration rate was reduced by 30% compared with a completely exposed glulam timber column.

(d) Glulam timber columns with inserted steel T-section in Figure 8.10 was classified according to DIN 4102 [8.7] as F30 and F60 if the width of the beam is at least 120 mm net and 2 x 55 mm gross, and at least 230 mm net and 2 x 110 mm gross, respectively. Kordina and Meyer-Ottens [8.1] recommended that for slimmer beams ($h/b \geq 4$) in class F60, the back of the T-section should be protected so that any charring due to this insert in a long fire would not cause any stability problem.

(e) Glulam timber column with steel support in Figure 8.11 was classified according to DIN 4102 [8.7] as F30 if certain minimum dimensions were satisfied. Kordina and Meyer-Ottens [8.1] indicate that the same fire resistance can be obtained for steel support brackets at least 10 mm thick as shown in Figure 8.14; provided that their height $\geq 2h/3$ (h = height of the beam) and their depth $\geq$ 100 mm and that the stress does not exceed 1.25 MPa.

(f) Concrete column with concrete restraint is also referred to in early investigations by Aarnio and Kallioniemi [8.3], [8.4]. The detail is shown in Figure 5.1b. The construction was loaded with the self-weight of the concrete column which, at the beginning of the test, corresponded to a load of 0.06 MPa. The testing time was 60 minutes. The temperatures between the side of the beam and the concrete restraint as well as that between the glulam timber beam and the concrete column were measured.

The results show that the surface temperature after 30 minutes exceeded 300°C and after 60 minutes it was as high as 700°C. The temperature in the gap between the concrete restraint and the side of the timber increased at different rates depending on the width of the gap, which at the beginning of the test was measured to be 0-5 mm at one side and 4-8 mm on the other. In the wider gap, the temperature was over 300°C after 20 minutes while it took a further 10 minutes before this temperature was reached in the narrower gap. After 60 minutes, the temperature in both gaps was above 800°C. The high temperatures at the end of the test were surprising because Aarnio and Kallioniemi did not observe any developed charring in the joint surfaces: "If the gap between the concrete and the timber in the restraint was greater than 5 mm, the timber surface had a yellowish brown colour which indicated that pyrolysis had commenced", see also Figures 5.2 and 5.3.

Kordina and Meyer-Ottens [8.1] have also indicated that, on the basis of analysis of the beam in Figure 5.1b, the estimated fire resistance would be more than 60 minutes provided that the concrete restraint has sufficient dimensions.

(g) Concrete column with steel restraint in Figure 5.1a was also tested by Aarnio and Kallioniemi. The load applied was the self-weight of the column which corresponded to a stress of 0.04 MPa. The testing time was 60 minutes. The temperatures in the steel plate and at the contact surface between the glulam beam and the concrete column were measured and the results were similar to that given in Figure 7.6 and section 8.2. Figure 5.2 shows the remaining cross-section after the test.

(h) Concrete column with inserted T-section, in principle according to Figure 8.10, was considered by Kordina and Meyer-Ottens [8.1] as giving the same or better fire resistance than glulam timber columns with inserted T sections, i.e., F30 or F60 depending on the width of the beam with the inserted element.

(i) Concrete column with concrete support as shown in Figure 8.12, according to Kordina and Meyer-Ottens, was in fire class F30 if stability was secured by wooden bolts and a steel angle at the top of the beam whereby the length of the bolt is at least 40 mm. Even a higher fire resistance is possible if the cross-section dimensions and the edge distances are increased.

(j) Concrete column with cast-in steel support brackets in Figure 8.13 was considered by Kordina and Meyer-Ottens as belonging to the same fire class, i.e., F30, as a corresponding installation of glulam timber column. The plate thickness must be at least 10 mm, the minimum height of the support is 2/3 the depth of the beam; with minimum length of 100 mm and the maximum stress being 1.25 MPa.

(k) Joint details for columns must be, according to Kordina and Meyer-Ottens, fire protected in order for fire classification to be possible.

8.3 Joints of Glulam Timber Secondary Beams to Glulam Timber Primary Beams

Aarnio and Kallioniemi have in their investigations, referred to earlier [8.3], [8.4], fire tested different fittings for fastening secondary beams to primary beams of glulam timber as shown in Figure 8.14. The design on the right hand side was tested with two different types of protections as shown in Figure 8.15. The temperatures at the contact surface between the steel and the underside of the beam were measured during the test; which lasted for 30 minutes. The results are given in Figure 8.16. All the joints were studied at two different load levels, i.e., half with the reaction of 15.4 kN and the other half with 5.2 kN, corresponding to stresses of 1.7 MPa and 0.6 MPa respectively at the beginning of the test. With the larger reaction, the displacement of the secondary beam relative to the primary beam was measured. The results are given in Figure 8.17.

Sections along the length of the secondary beam, for fitting types K1-K3 after the test are shown in Figure 8.18. For fitting type K4, the underside of the beam ends were uncharred after the test.

Aarnio and Kallioniemi stated that fitting type K2 had the worst fire resistance and their loadbearing capacity was exhausted when the test was interrupted, i.e., after 30 minutes. Kordina and Meyer-Ottens [8.1] also found, through tests, that the fire resistance of a similar fitting in Figure 8.11 was 34-36 minutes. The thickness of the plate was 10 mm and the stress was 1.25 MPa. Corresponding figures used by Aarnio and

Kallioniemi were 5 mm and 1.7 MPa respectively. Fastenings as shown in Figure 8.11 are classified as class F30 according to DIN 4102 if the minimum dimensions in the table included are satisfied.

Aarnio and Kallioniemi considered that fitting type K1 performed better. However, the fitting has deformed significantly after the test, which was not the case for other types of fitting. Hence its fire resistance was not considered to be in class F30.

Kordina and Meyer-Ottens, based on their test results [8.2], also stated that fittings with support brackets type BMF, see Figure 8.19, can be taken as class F30 without any protective measures. Certain minimum dimensions, however, must be met and these are included in Figure 8.19. Their results are summarised in Figure 8.20. The failure mode, according to Kordina and Meyer-Ottens, involved the deformation of the nails in the secondary beam which subsequently increased the reaction transferred through bearing between the beam and the support. Successive deformations of the beam support and the charring at the end of the beam resulted in the beams sliding off their supports.

Aarnio and Kallioniemi finally stated that for the two fire protection systems considered, i.e., types K3 and K4, the simpler alternative K3 gave approximately the same fire resistance as the unprotected type K1. Of the four alternatives investigated, they considered that only fastening with fire protections in accordance with Figure 8.15, i.e., type K4, corresponded to class F30.

8.4 Beam Joints and Ridge Joints

Ahlen and Mansson [8.5] have fire tested two different types of beam joints at two load levels, i.e., corresponded to 65% and 100% of permissible loads for an ordinary load situation. Testing periods varied from 16 to 42 minutes depending on when the load capacity was considered to be exhausted. During the tests, temperatures at points of interest as well as the relative displacements of the beams were recorded. The relationship between the penetration rate and the proof stress was discussed in Chapter 4. As far as the comparison between the two joint alternatives is concerned, it is noted that the bolted connection had better fire performance than the fitted sleeve.

Based on the results from Ahlen and Mansson's investigation, it was stated in [8.8] that the bolted joint without fire protection as shown in Figure 4.4, alternative 1, can fulfill the requirements of class B30 if the load during the fire was at the most 65% of the permissible for an ordinary load situation. At a higher load utilization, some form of fire protection is required, e.g., 30 mm thick mineral wool. For the joint with fitted sleeve as shown in Figure 4.4 alternative 2, the same source stated that class B30 can be achieved with fire protections provided that the load does not exceed 65% of the permissible.

Girder connections, partially protected with a 30 mm thick cover plate as shown in Figure 8.21, based on the investigations of Kordina and Meyer-Ottens [8.1] is classified as class F30 according to DIN 4012 [8.7]. The stress must not exceed 1.6 MPa. Failure was described as a shear fracture in the obliquely cut and weakened end of the beam, which was weakened due to the effects of fire. Charring took place both at the outside and inside of the beam as a result of the metal details being heated.

Kordina and Meyer-Ottens [8.1], [8.2] have also fire tested ridge joints as shown in Figure 8.22 with and without fire protection. In Table 8.4, they presented the results of different tests.

The loads in these tests corresponded to a horizontal force at the ridge joint of 69.3 kN and a vertical force of 40 kN except for test number 4 where the corresponding figures were 91.5 and 52.8 KN respectively. Based on these tests, it was stated in [8.1] that the fire class was F30-B for partially protected fittings and class F90-B for fittings protected with wooden plugs, plates, and packing, according to the specifications in Figure 8.23. A certain limit on visible deflection was, however, applied to both cases.

[8.1] = [3.3]

[8.2] = [3.4]

[8.3] = [1.14]

[8.4] = [1.15]

[8.5] = [4.5]

[8.6] = [4.4]

[8.7] = [4.15]

[8.8] Johannesson, B. 1979. Glulam Timber Handbook. Swedish Glulam Timber Association. Stockholm.

9 ANALYTICAL METHODS

Analytical solutions for determining the actions and loadbearing capacities of buildings during fire are available and have been used extensively in practice, particularly for steel structures [9.1], [9.2]. For prestressed concrete structures, corresponding methods of calculation were developed [9.3] but they have not been used extensively in common practice.

On the whole, the fire resistance of different structural elements of timber, especially glulam timber, are readily calculable; while the application of analytical methods for light and composite constructions remains a remote possibility.

Barthelemy and Kruppa [9.4] presented a method which estimates the fire resistance of simple connection details with unprotected metal parts, e.g., a bolted timber joint as illustrated in Figure 9.1.

Hertz [9.5], [9.6] has further developed the method whereby the temperature of the metal details was calculated according to the same principle as that applied to large steel structures [9.1]. When this temperature exceeded 280°C, it was assumed that the charring of the timber takes place from the covered surfaces at a constant rate (0.6 mm/min), which is the same as that for directly exposed timber surfaces. The load capacity of the joint at a certain time thereafter is estimated from the uncharred timber cross section and the capacity of the metal details at the temperature at that time.

The method just described can be further developed by taking into account the rate of heat penetration at the adjoining surfaces; which depends on the load applied, the direction of heat energy and the temperature etc.

[9.1] = [1.1]

[9.2] = [1.2]

[9.3] Anderberg, Y., Pettersson, O, Thelandersson, S. and Wickstrom, U. Design of Concrete Structures for Fire Resistance. Handbook under preparation.

[9.4] Barthelemy, B. and Kruppa, I. 1978. Resistance au feu des structures Beton-Acier-Bois (in French, title translates to - Fire resistance of concrete, steel and timber). Editions Eyrolles. Paris.

[9.5] Hertz, K. 1983. Brandteknisk dimensionering af traekonstruktioner (in Danish, title translates to - Design of timber structures for fire resistance). Technical Institute of Denmark, Institute for House Constructions, lecture notes no. 64. Lyngby.

[9.6] Hertz, K. 1981. Beregning af traesamlinger under brand (in Danish, title translates to : Analysis of timber structures during fire). Brandvaern No. 6-1981. Copenhagen.

10 STANDARDS

In many countries, the treatment in regulations and standards for fire design of joint details and connections in timber structures is either very brief or non-existent.

In Sweden, PFS 1984:1 'The loadbearing capacity of building details during fire' [10.1] gives the following recommendations:

"The characteristic loadbearing capacity of connections made from steel (nails, bolts and gussets) are assumed to decrease at elevated temperatures in the same way as those for steel structures according to the above. The connections shall be assumed to have a temperature development as if unprotected unless a special investigation shows a more favourable development. Furthermore, it is assumed that a transfer of load between the joint components and the charred timber cannot be taken into account. For the calculation of the characteristic loadbearing capacity of the joint components, one accepts that the strengthfor an ordinary load case (without the temperature being raised) according to SBN Chapter 27 can be increased by 50%."

In Denmark, the Danish Engineering Association Standards for Timber Structures, DS 413 [10.2] assumes that the heat transmitting metal parts are protected against heat damage; for example, by using a wooden plug or other types of insulation.

Norwegian Standard NS 3478 [10.3] specifies that components which are in direct contact with timber must be properly insulated such that their temperature does not exceed 300°C during the fire.

Finnish Regulations [10.4] specify that metal parts which form part of a loadbearing component in timber construction shall be so insulated that they can achieve the same fire resistance as the structure as a whole. Timber, suitable boards or mineral wool of sufficient thickness can be used as insulation. The materials which are in direct contact with the timber must be so insulated that the temperature does not exceed 300°C during the fire.

German Standards [10.5] give explicit regulations on how joint details shall be formed or shaped in order to fulfill the requirements in different fire classes. Several of these details have been given in Chapters 4, 7 and 8.

British Standard BS 5268 point out that metal parts which heat up during fire can cause local charring and therefore may lead to collapse of the structure. Thus these parts must be protected either by burying them so deeply into the timber such that they lie completely within the effective cross-section according to Figure 10.1 or by using suitable insulation material, e.g., timber board, asbestos-cement board or equivalent. For beam constructions in class B30, it is specified that when 1 mm steel plate is used, the ceiling must be provided with at least 20 minutes fire resistance; for instance that of 12 mm particle board. However, this requirement is no longer necessary when protection for the beam is provided, and that it is made of steel plates 3 mm thick.

The Australian Standards [10.7] follow BS5268 to some extent, but also give certain guidelines for the fire resistance of different types of joint details, see Table 10.1.

Type of joint	Fire Resistance (minutes)
Toothed plate	5
Intermediate plate*	10
Bolts	20
Nails**	30

*Split ring or shear plate
** Nailed joint of the same fire resistance as solid timber

Table 10.1 Typical fire resistance of unprotected timber joints according to Australian Standards [10.7].

[10.1]State Planning Commission. 1984. SBN approval list for the strength of structural elements during fire. PFS 1984:1. Liber Publisher. Stockholm.

[10.2] = [2.6]

[10.3]Norwegian Standards Institute. 1981. NS 3478 Brannteknisk dimensjonering av bygningskonstruksjoner (in Norwegian, title translates to - Fire design of timber structures). Oslo.

[10.4]Minister of Internal Affairs (issued by). 1977. Finland Building Regulations, part 5. 1977. Barande och avskiljande konstruktioners brandstabilitet (in Finnish, title translates to - Fire stability of loadbearing and partition constructions). Helsinki.

[10.5] = [4.15]

[10.6]British Standards Institution. 1978. BS 5268: Code of practice for the structural use of timber, part 4: method of calculating fire resistance of timber members. London.

[10.7]Standards Association of Australia. Draft Australian Standard DR 83201, Timber engineering code, part 4: fire resistance of timber structures.

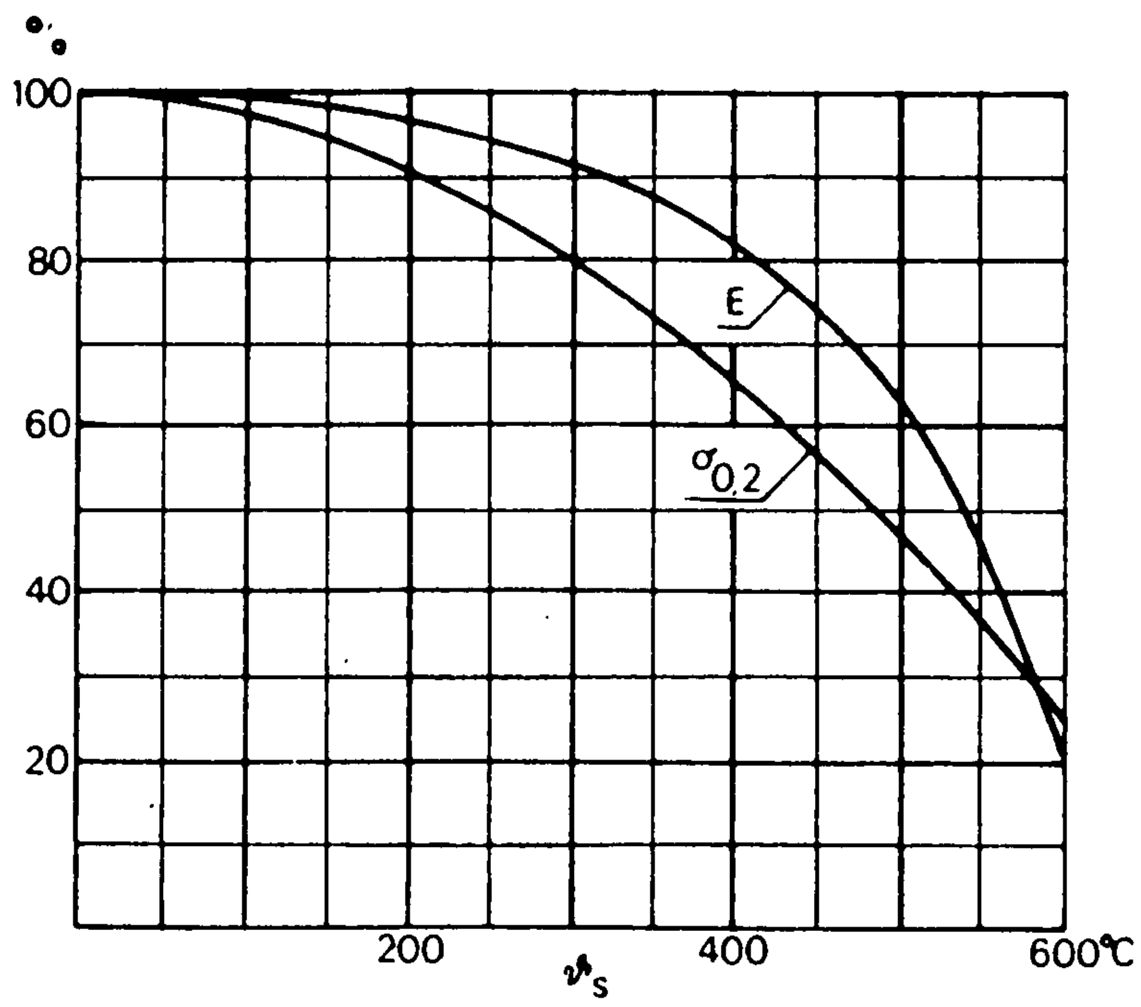


Figure 1.1: 0.2-limit and modulus of elasticity as a function of the steel temperature for mild steel [1.1].

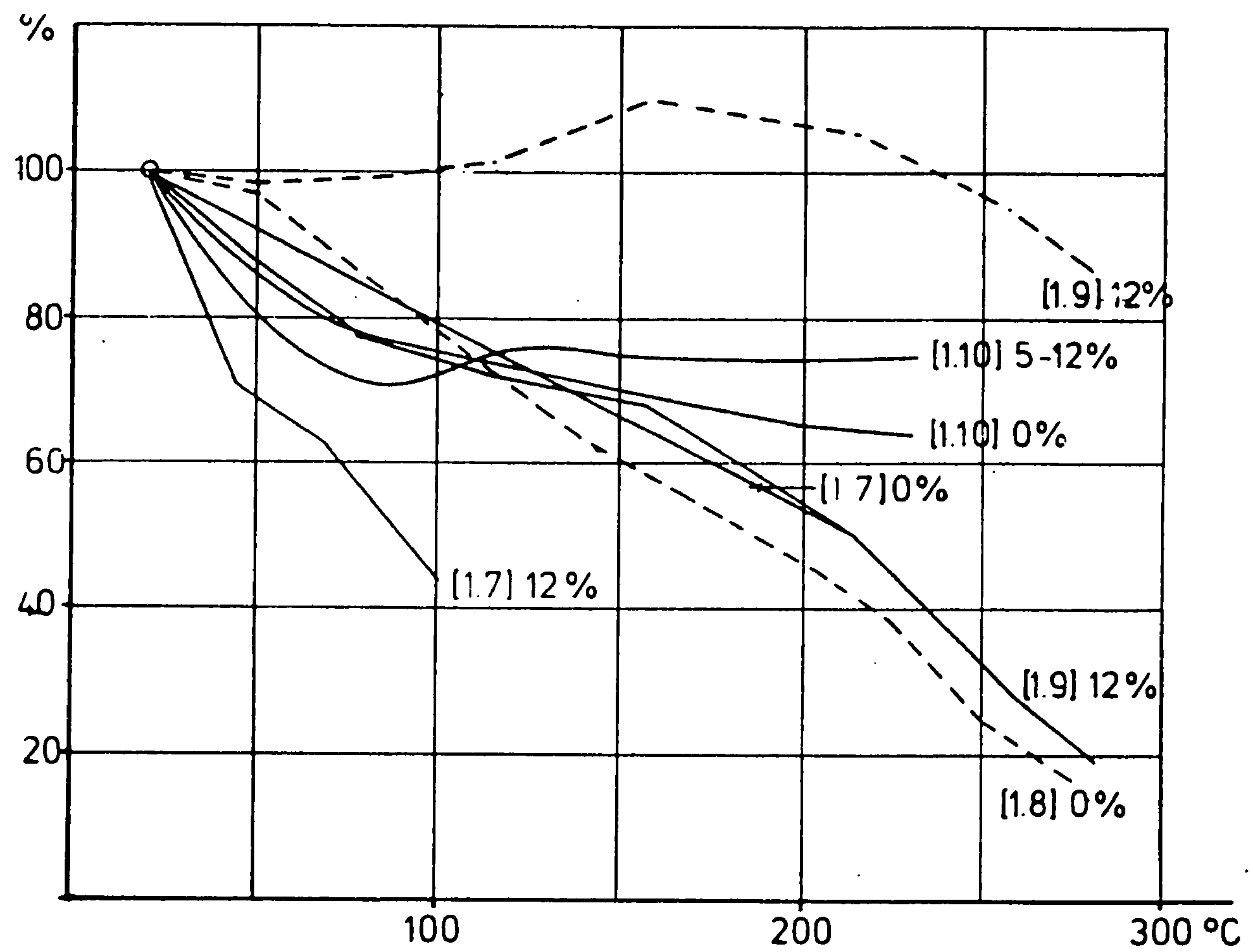


Figure 1.2: Relative compressive strength parallel to the grain direction as a function of temperature for Douglas fir [1.8], [1.9], spruce [1.7], [1.10] and pine [1.10].

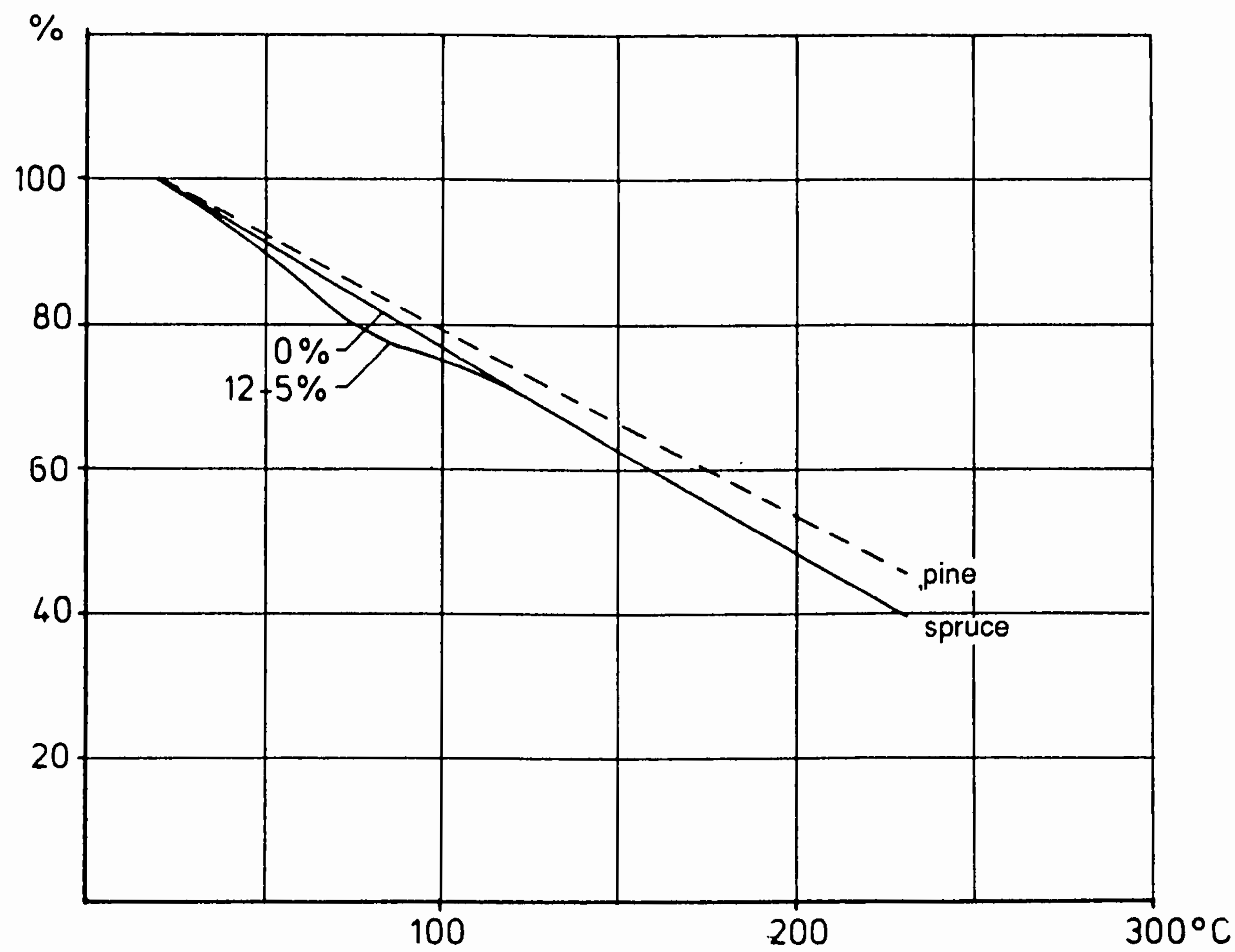


Figure 1.3: Relative shear strength as a function of temperature for spruce and pine [1.10].

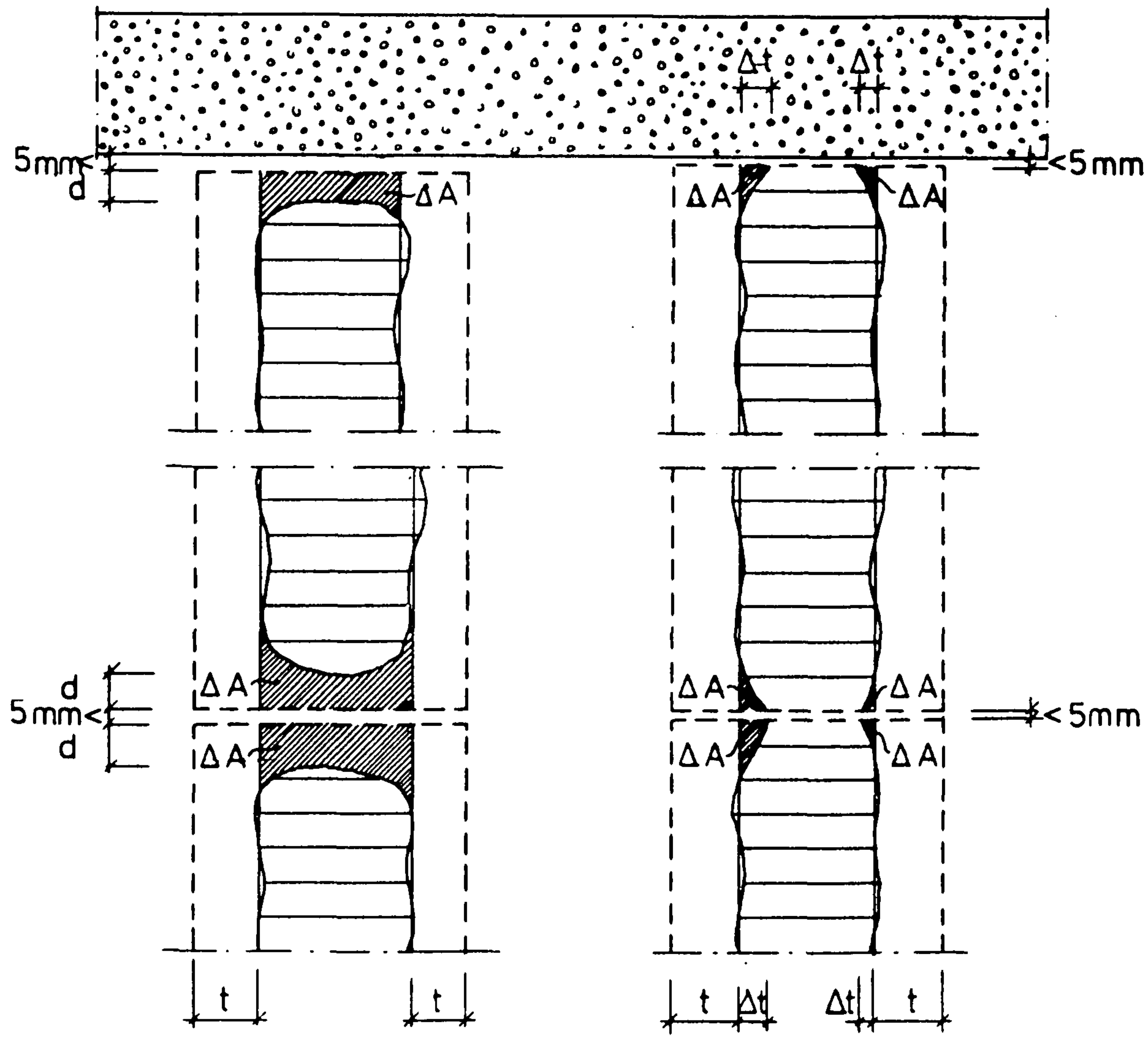


Figure 3.1: The effect of the width of a gap, between two glulam timber beams or between a glulam timber beam and a lightweight concrete slab, on the extent of the charred zone in the vicinity of the gap.

t = average horizontal depth of penetration
Δt = increase in penetration for a gap width less than 5 mm
d = vertical penetration at the gap greater than or equal to 5mm
ΔA = increased charred zone at the gap (per beam).

Test No.	Width of beam (mm)	Duration of test (min)	t (mm)	Gap No.	Width of gap (mm)	Δt (mm)	d (mm)	[ΔA (mm2)
R1	90	30	18	1	3	7	0	30
				2	4	7	0	50
				3	6	(45)	7	280
				4	8	(45)	17	600
R2	140	30	20	1	2	3	0	60
				2	3	5	0	70
				3	7	(70)	5	550
				4	8	(70)	11	920
R3	140	60	38	1	3	6	0	70
				2	3	8	0	40
				3	5	(70)	10	550
				4	7	(70)	28	1180
R1	90	30	18	1	2	2	0	30
				2	4	6	0	120
				3	6	(45)	5	450
				4	8	(45)	6	360
R2	140	30	20	1	2	2	0	20
				2	3	5	0	60
				3	6	21	0	210
				4	8	(70)	1	260
R3	140	60	38	1	2	5	0	40
				2	4	7	0	50
				3	6	(70)	8	500
				4	8	(70)	16	700

Table 3.1: Penetration depths for different gap widths between two glulam timber beams and a glulam beam and a lightweight concrete slab according to references [3.1], [3.2].

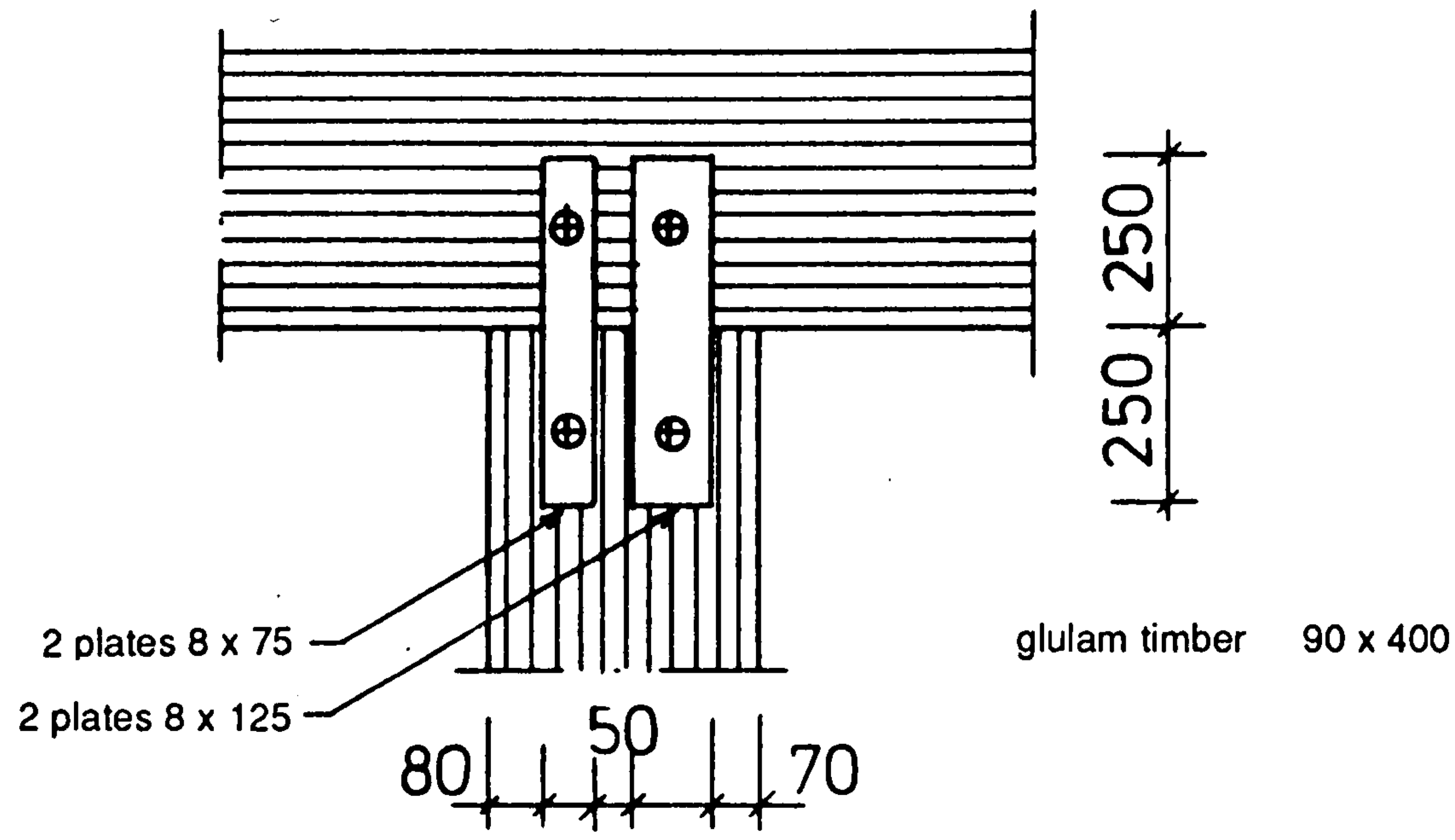


Figure 4.1: Fire tested beam-to-column joints with flat plates and bolts [4.1], [4.2]

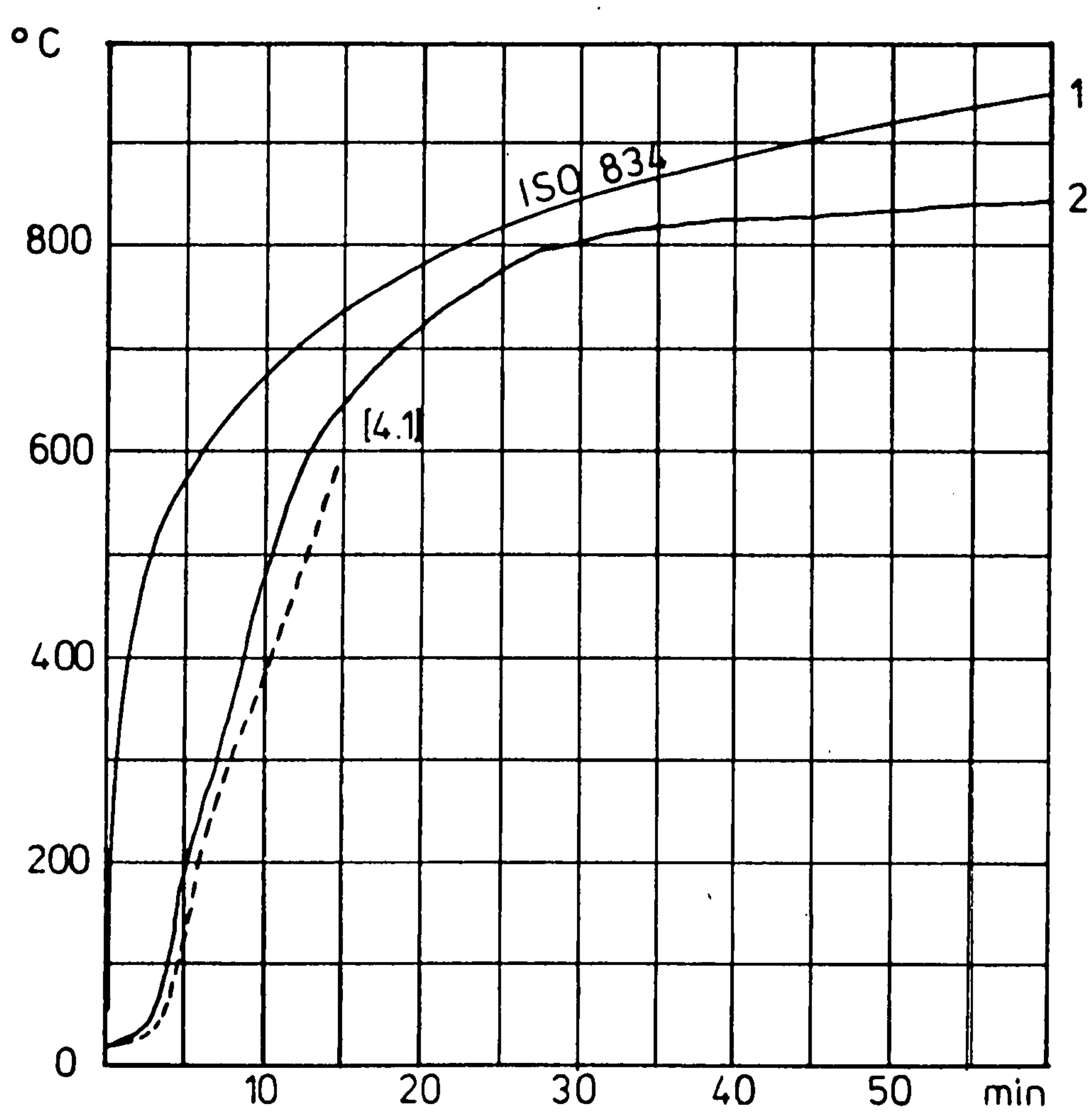


Figure 4.2: Temperature of flat plates during fire testing of beam-to-column joints in Figure 4.1 [4.1], [4.4]

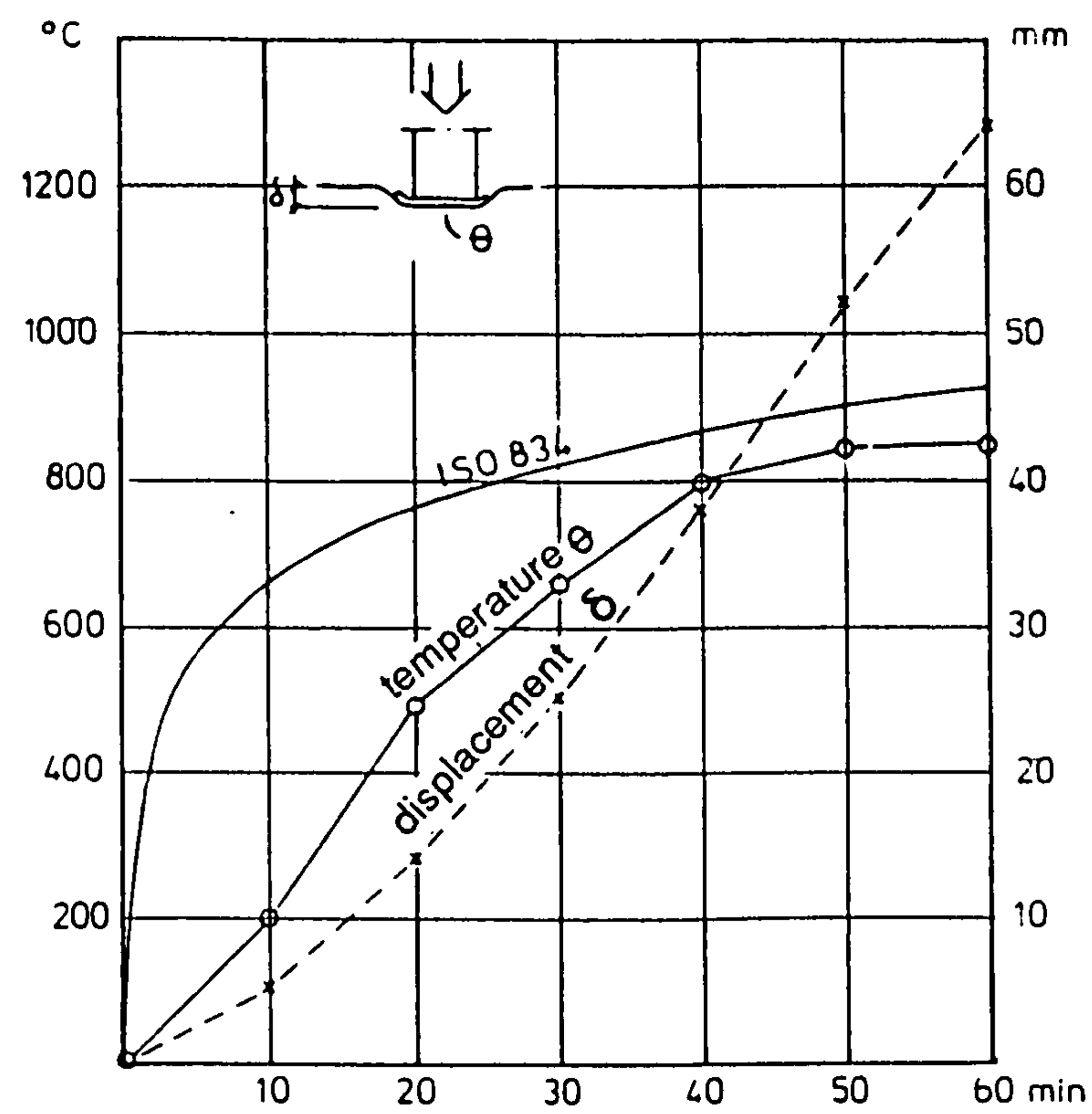


Figure 4.3: Temperature at the contact surface between the steel plate and the timber, and the displacement of the steel plate during the fire test in accordance with ISO 834.

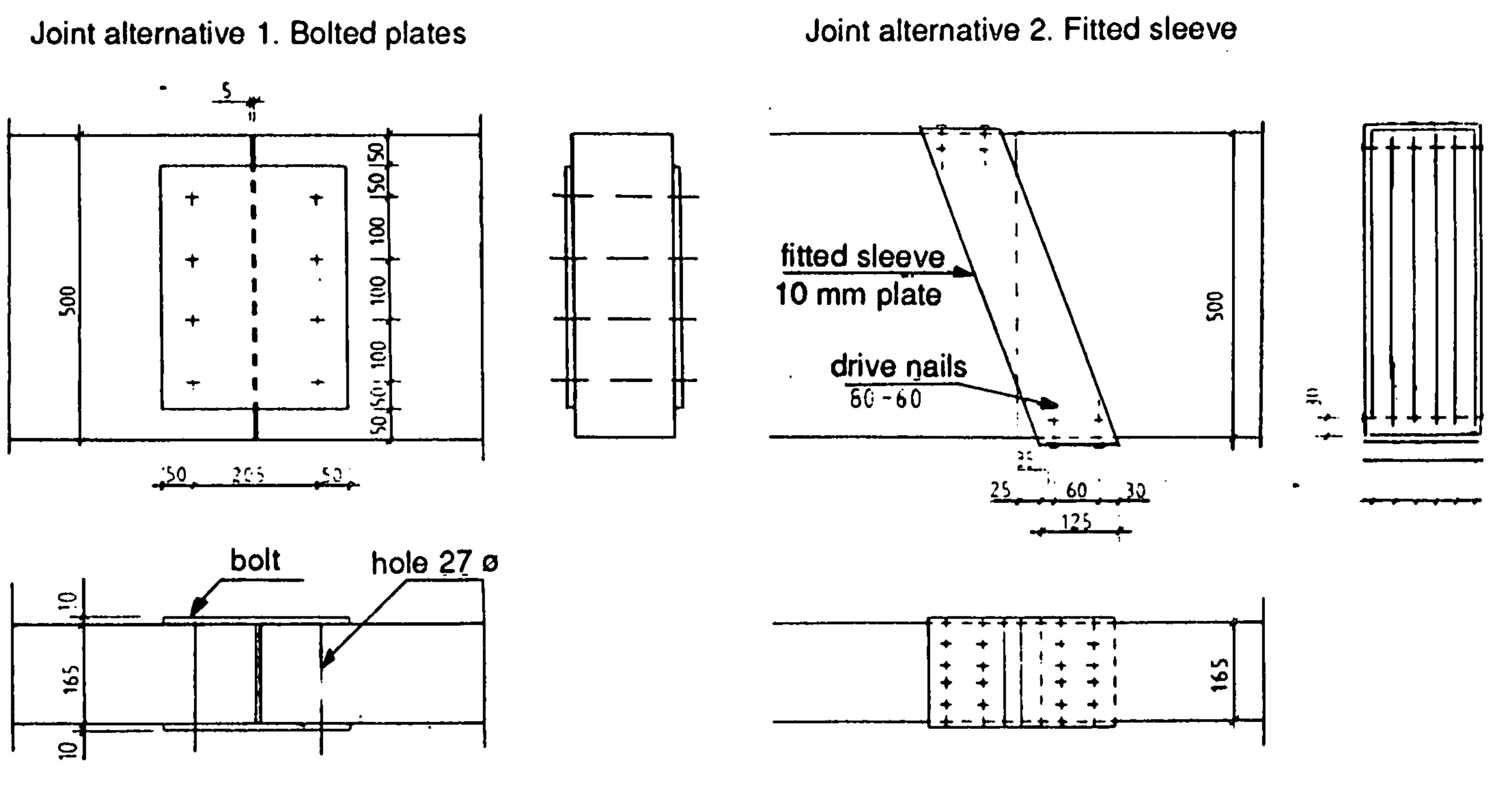


Figure 4.4: Fire tested beam joints with bolted plates (alternative 1) and fitted sleeve (alternative 2). Two load levels were investigated, which corresponded to 65% and 100% respectively of the permissible load [4.5].

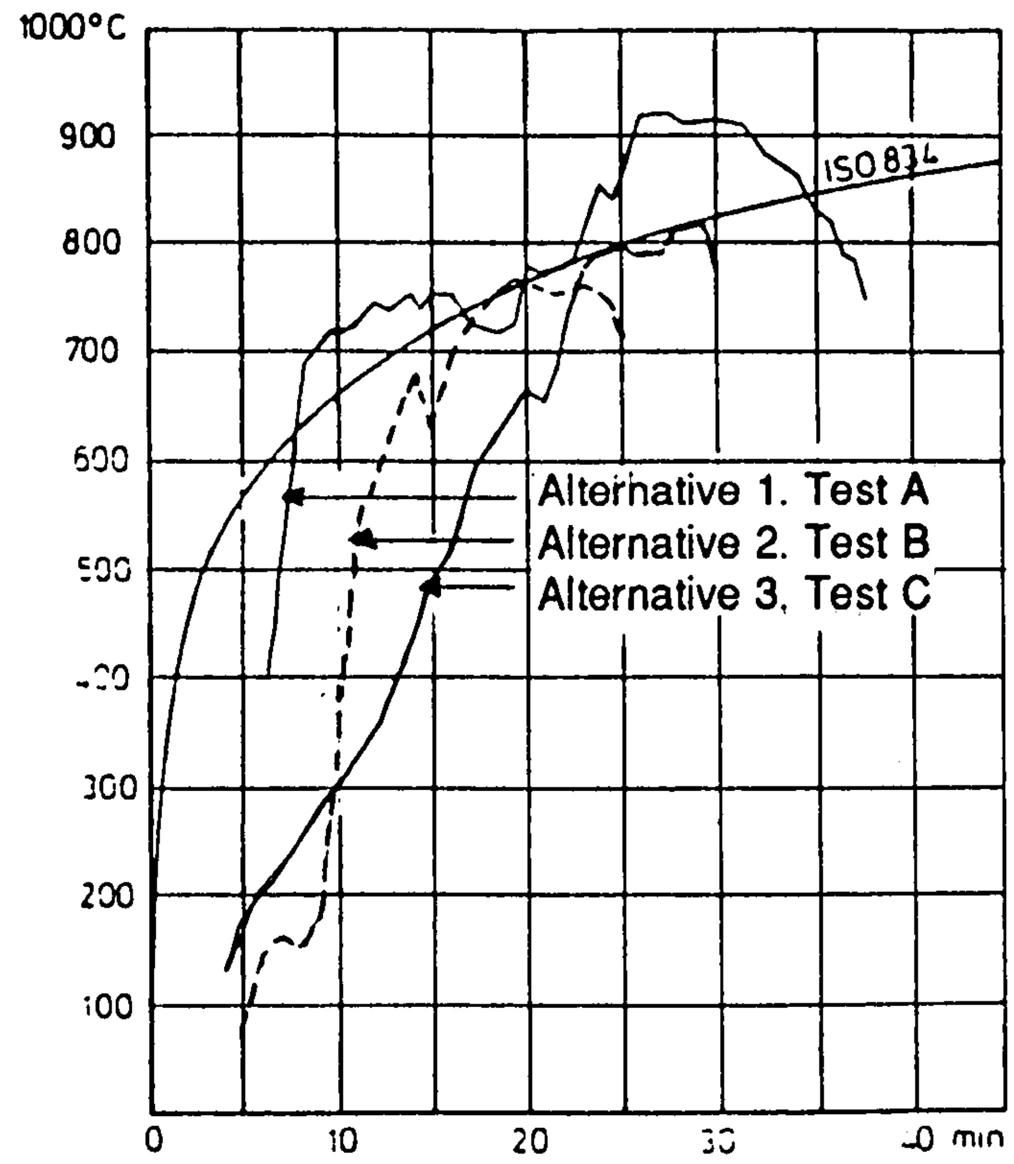


Figure 4.5: Temperature at the contact surfaces between glulam timber beam and the joint plates and side surfaces respectively as a function of time [4.5].

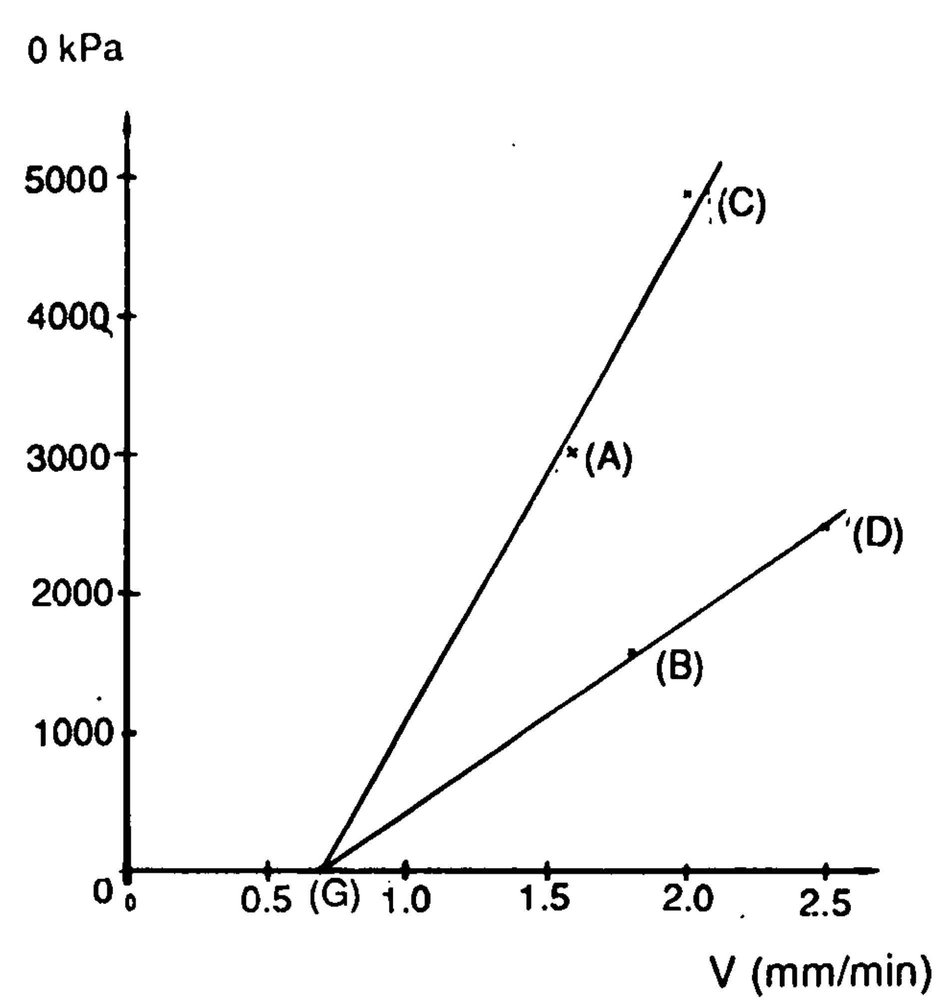


Figure 4.6: Empirical relationship between proof stress and the charring rate for the joints in Figure 4.4 [4.5].

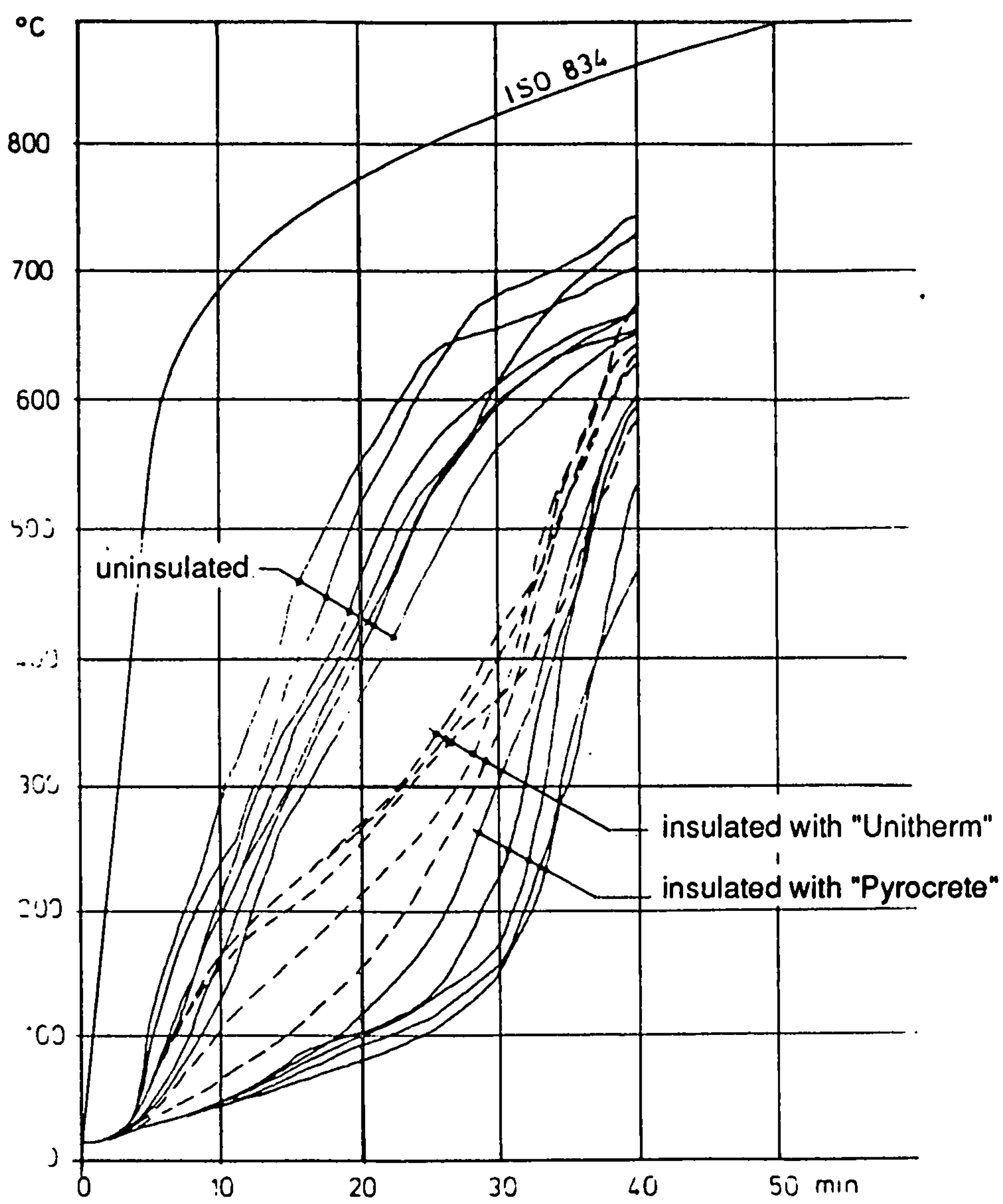


Figure 4.7: Temperature development in steel plates with or without insulation under fire tests in accordance with ISO 834 [4.6], [4.8].

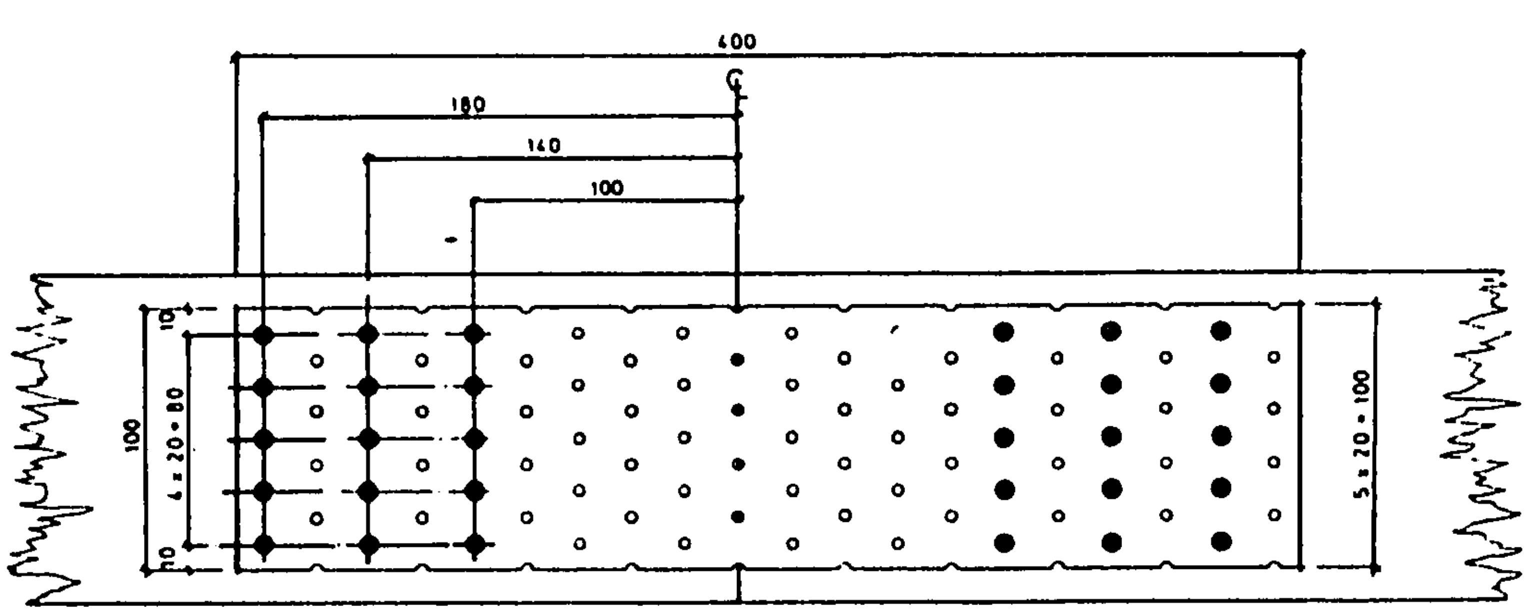


Figure 4.8: Fire tested timber joint with two nailed plates of 2.5 x 100 x 400 and 4 x 15 nails 40/60 [4.9].

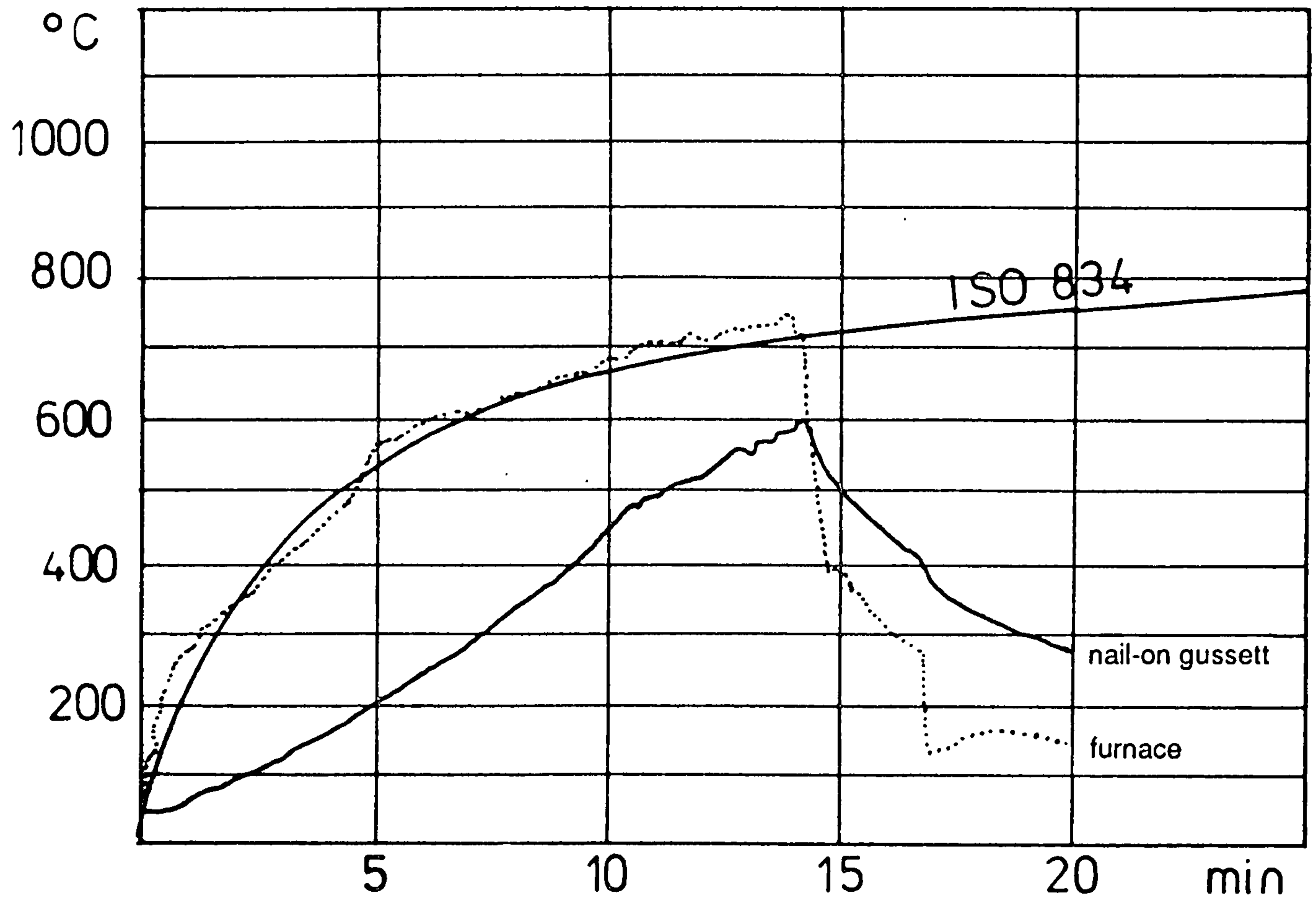


Figure 4.9: Temperature development in nail-on gussets as shown in Figure 4.8 [4.9].

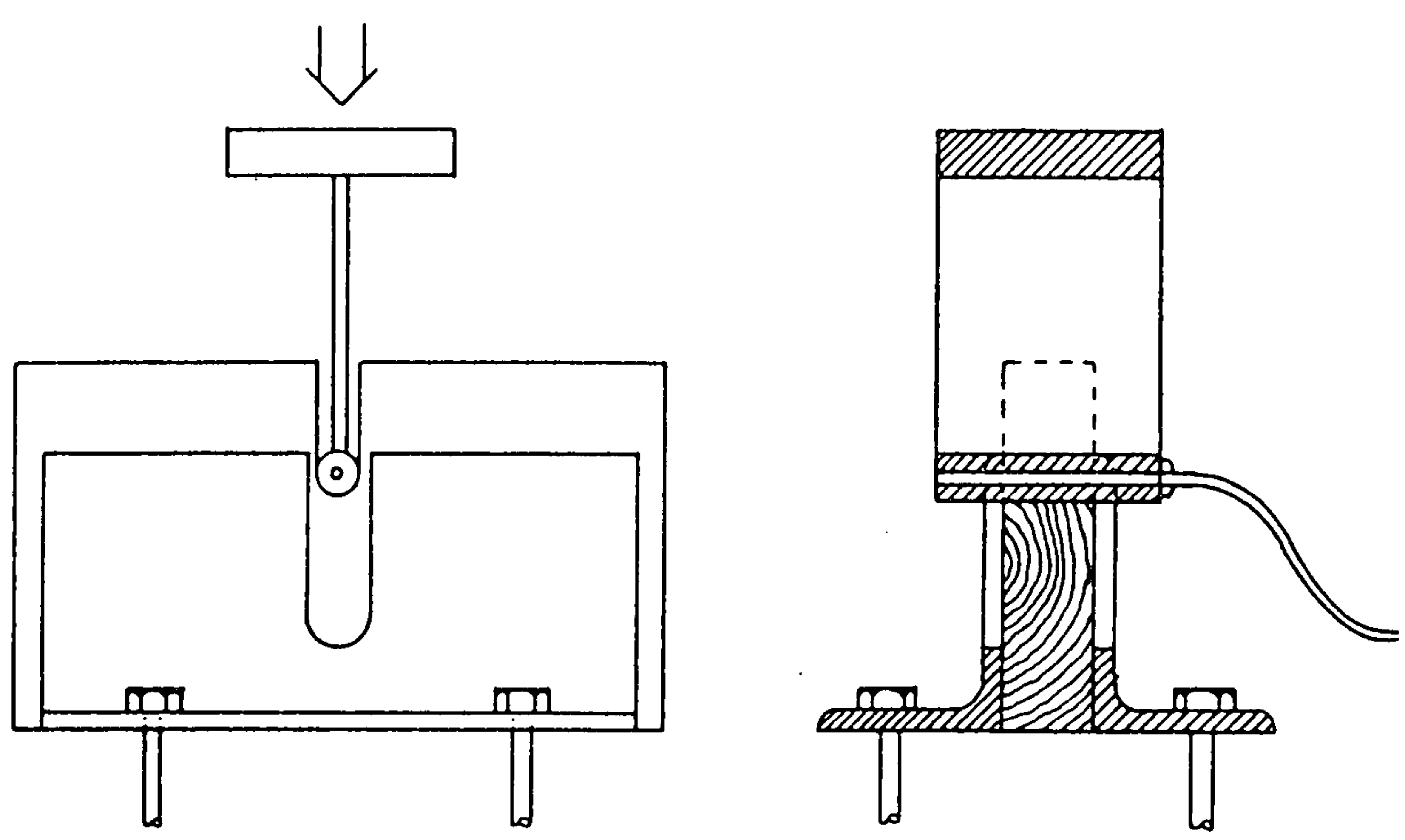


Figure 4.10: Test arrangements with an electrically heated dowel according to [4.10], and based on a photograph.

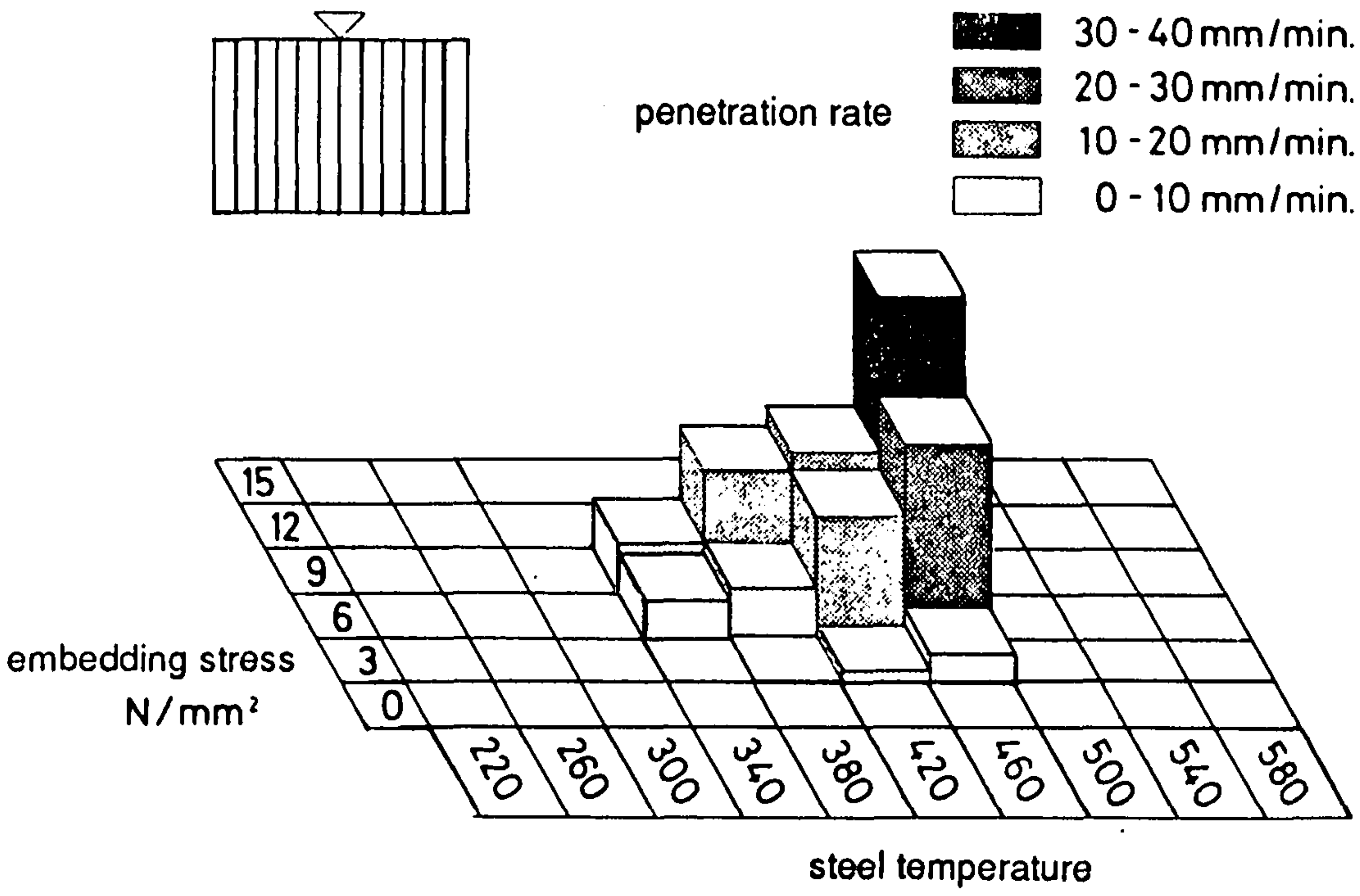


Figure 4.11: Penetration rates at force-fibre angle 0°. Each column represents average initial rate measured by four tests, regardless of the two dowel diameters [4.10].

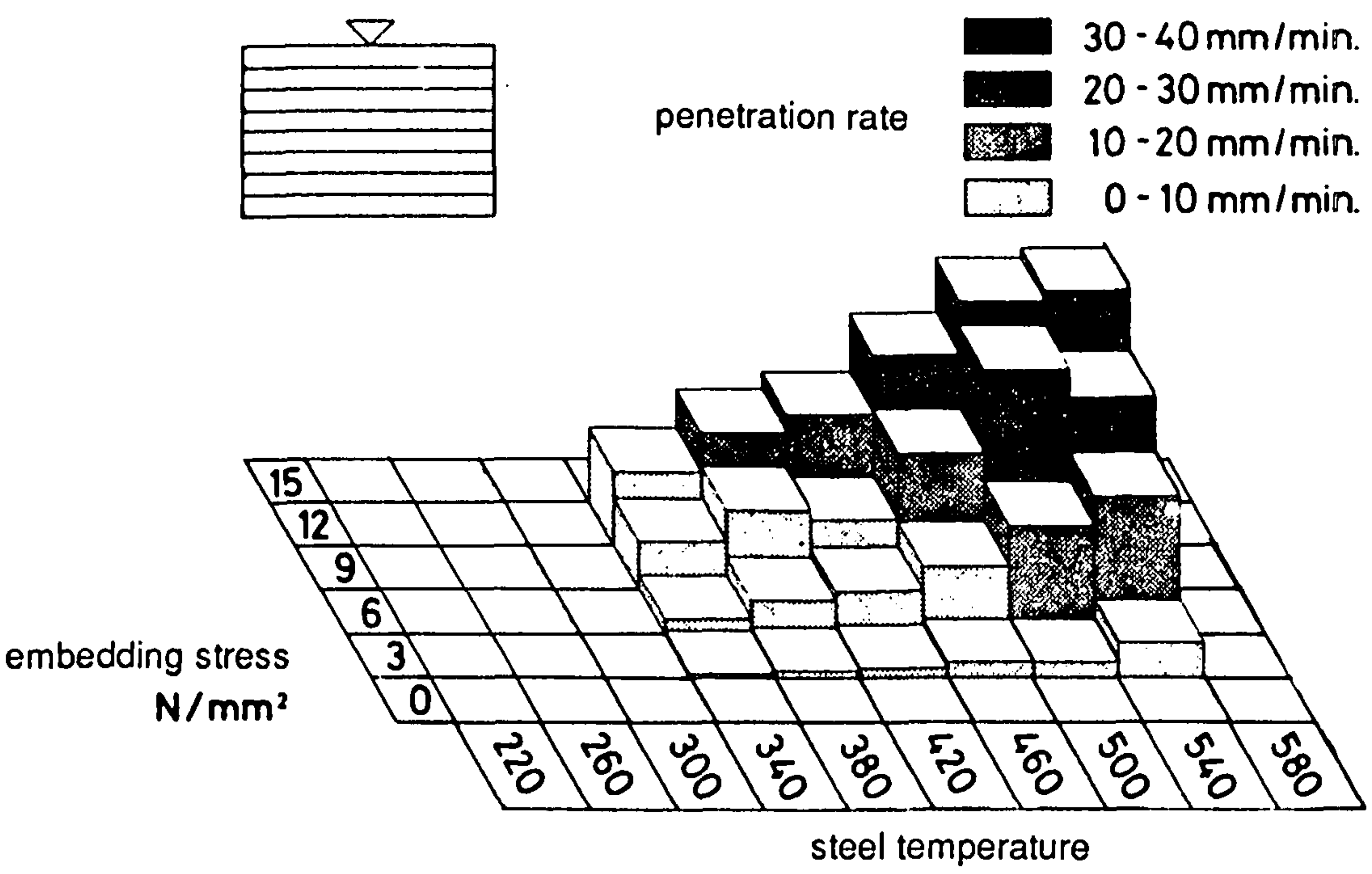


Figure 4.12: Penetration rates at force-fibre angle 90°. Each column represents average initial rate measured by four tests, regardless of the two dowel diameters [4.10].

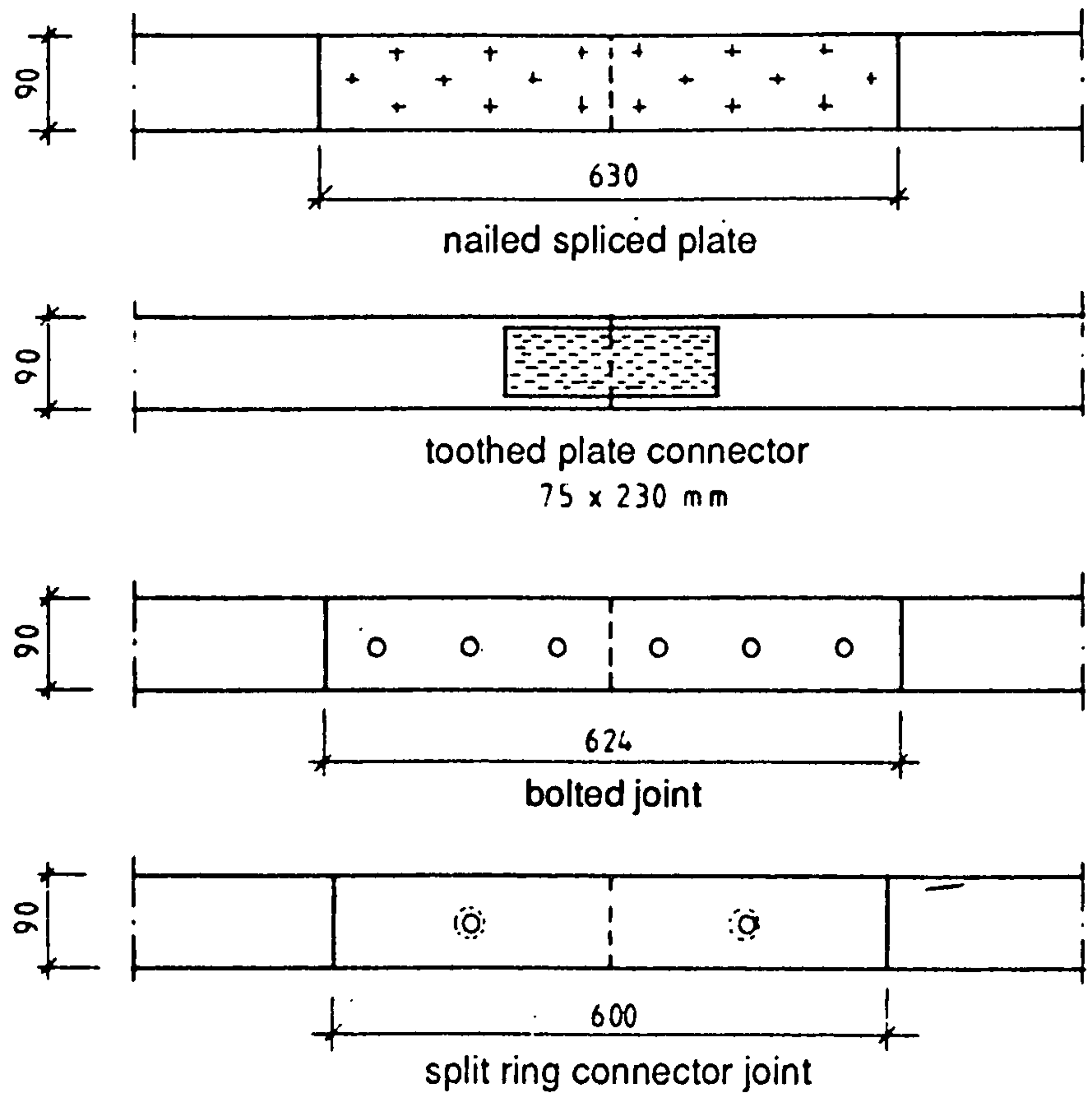


Figure 4.13: Timber joint samples under tension in Australian fire tests [4.12].

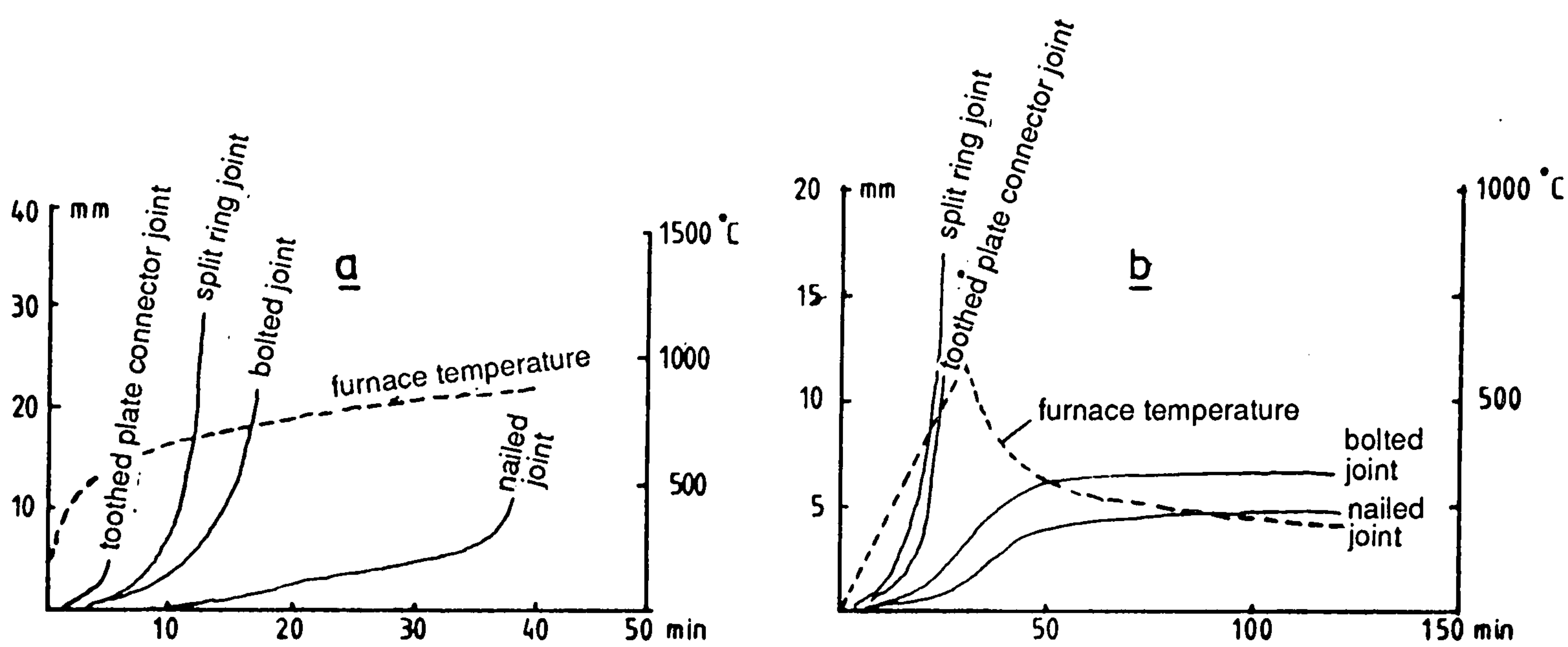


Figure 4.14: Deformation of timber joints under tension, as shown in Figure 4.13, in fire tests
a) standard fire condition,
b) mild fire condition.

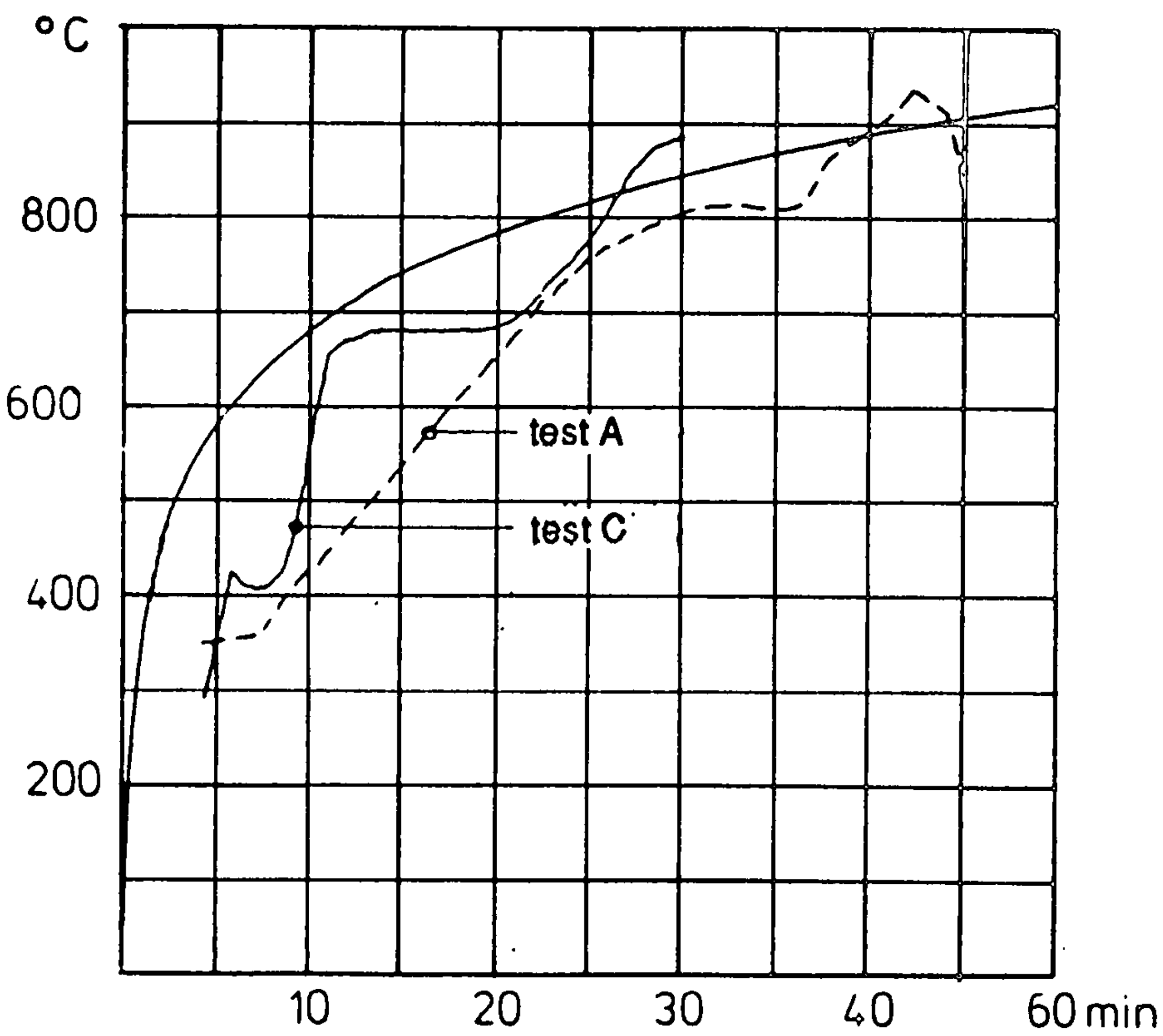


Figure 4.15: Temperature of the bolts 10 mm from the underside of the plate as a function of time [4.5].

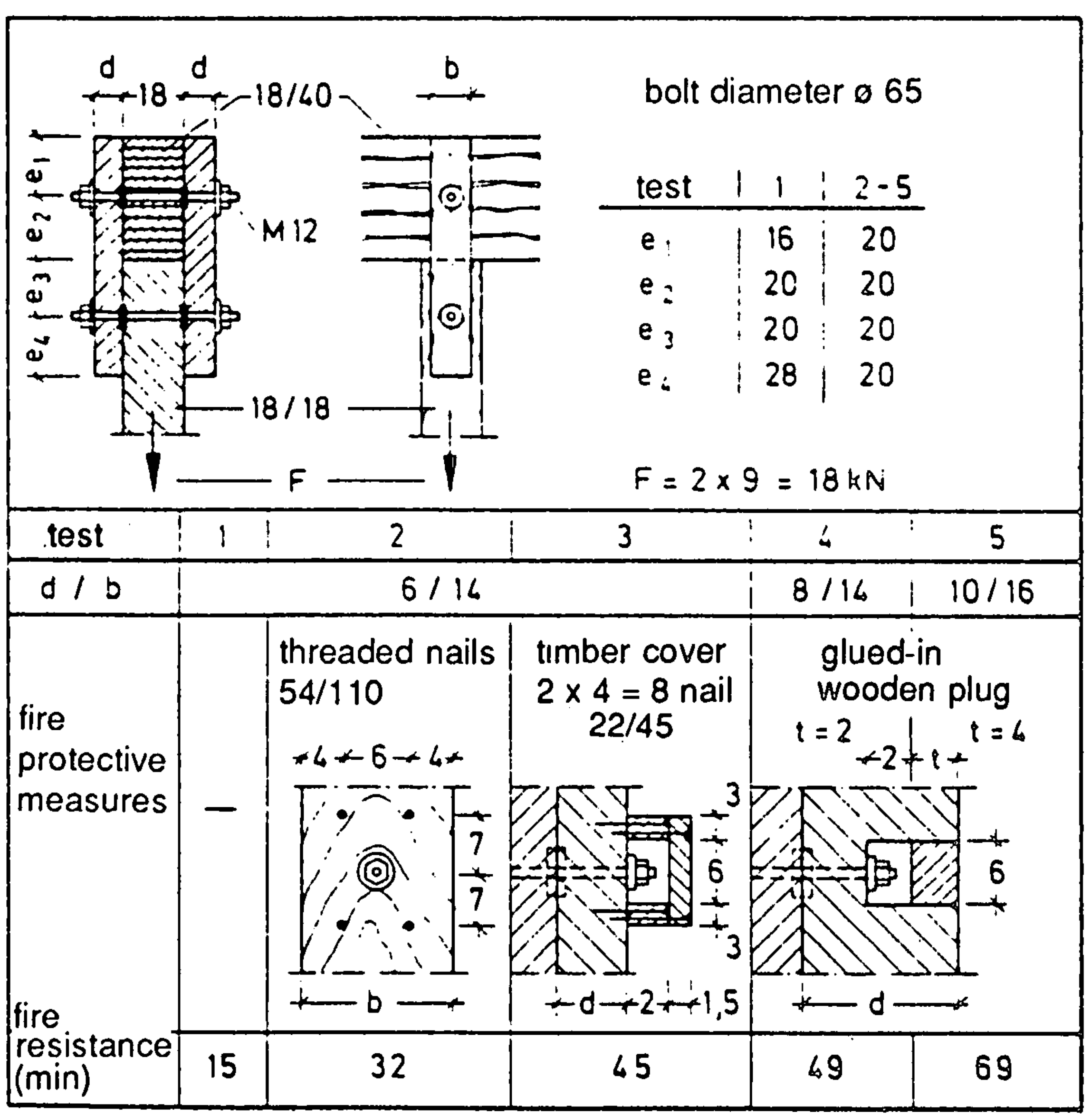


Figure 4.16: Fire tested beam or beam-to-column bolted joints in structural framework.. External bolted timber joints with washers [4.14].

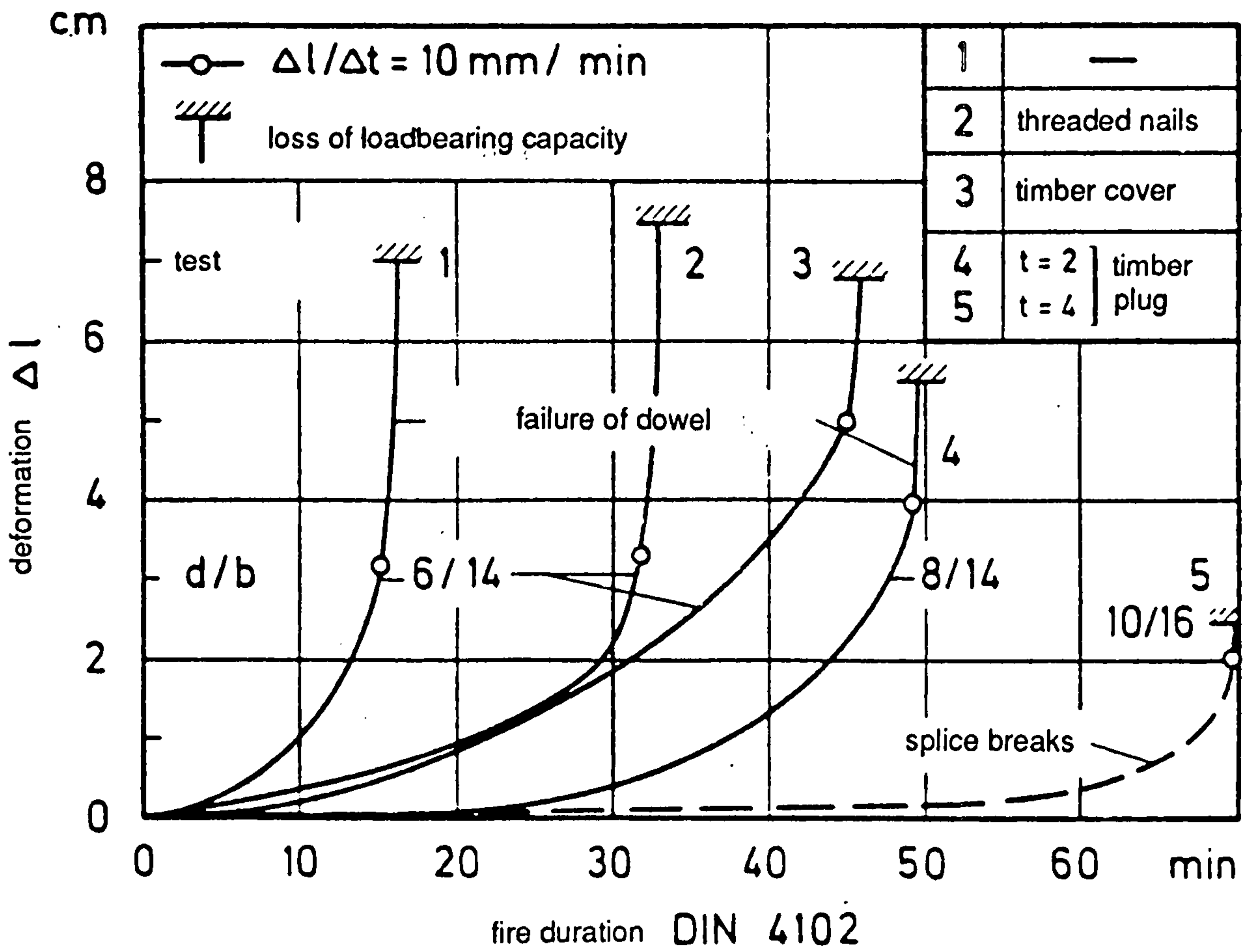


Figure 4.17: Deformation of bolted joints in Figure 4.16 during fire tests [4.14].

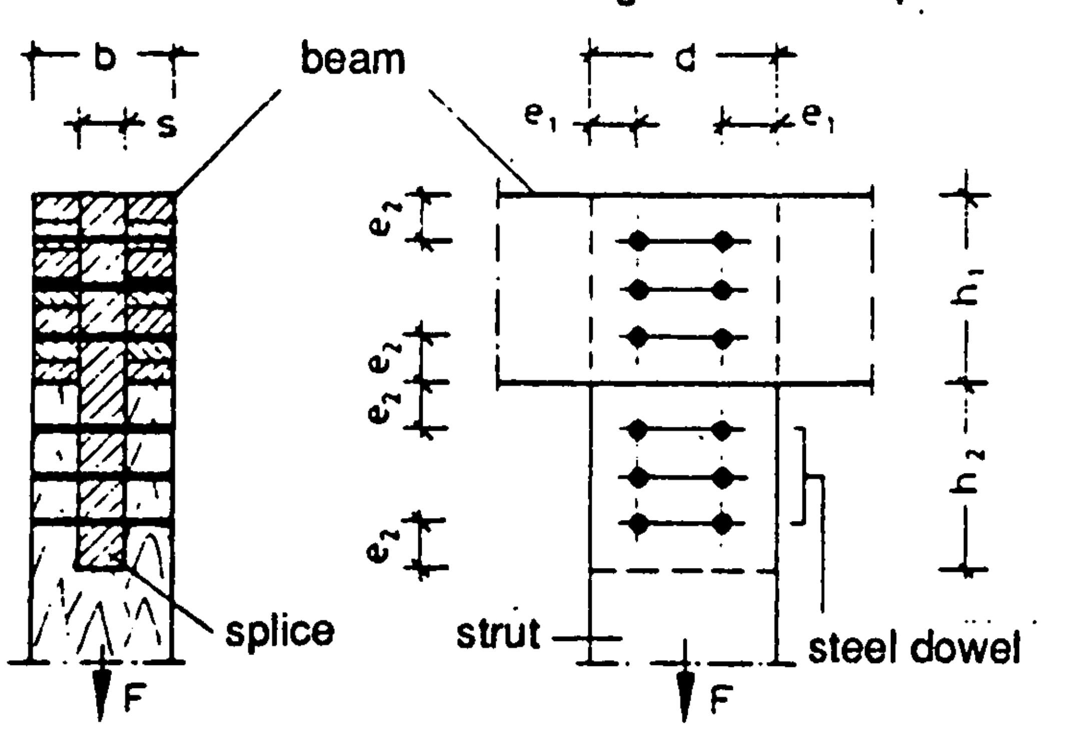
line	construction criteria for building components of timber (dimensions in mm)	minimum dimensions for F30-B
1 1.2	symmetric connection with internal splice plate of solid timber fixed with steel dowels according to DIN 1052 part 1 	b ≦ 150 mm d ≧ 140 mm s ≦ 40 mm ≦ b/3 h_1 ≧ 300 mm h_2 ≧ 300 mm e_1 ≧ 45 mm e_2 ≧ 80 mm n ≧ 6
2	symmetric connection with internal splice of construction plywood "AW 100" according to DIN 68 705 Part 5 fixed with steel dowels according to DIN 1052 part 1 notes see line 1.2	b ≧ 120 mm d ≧ 120 mm s ≧ 30 mm ≦ b/4 h_1 ≧ 260 mm h_2 ≧ 260 mm e_1 ≧ 40 mm e_2 ≧ 70 mm n ≧ 6 with F ≦ 0.5 permissible F

Figure 4.18: Fire tested beam-to-column joints in structural framework. Joints of inserted timber or plywood plates with dowel fittings.

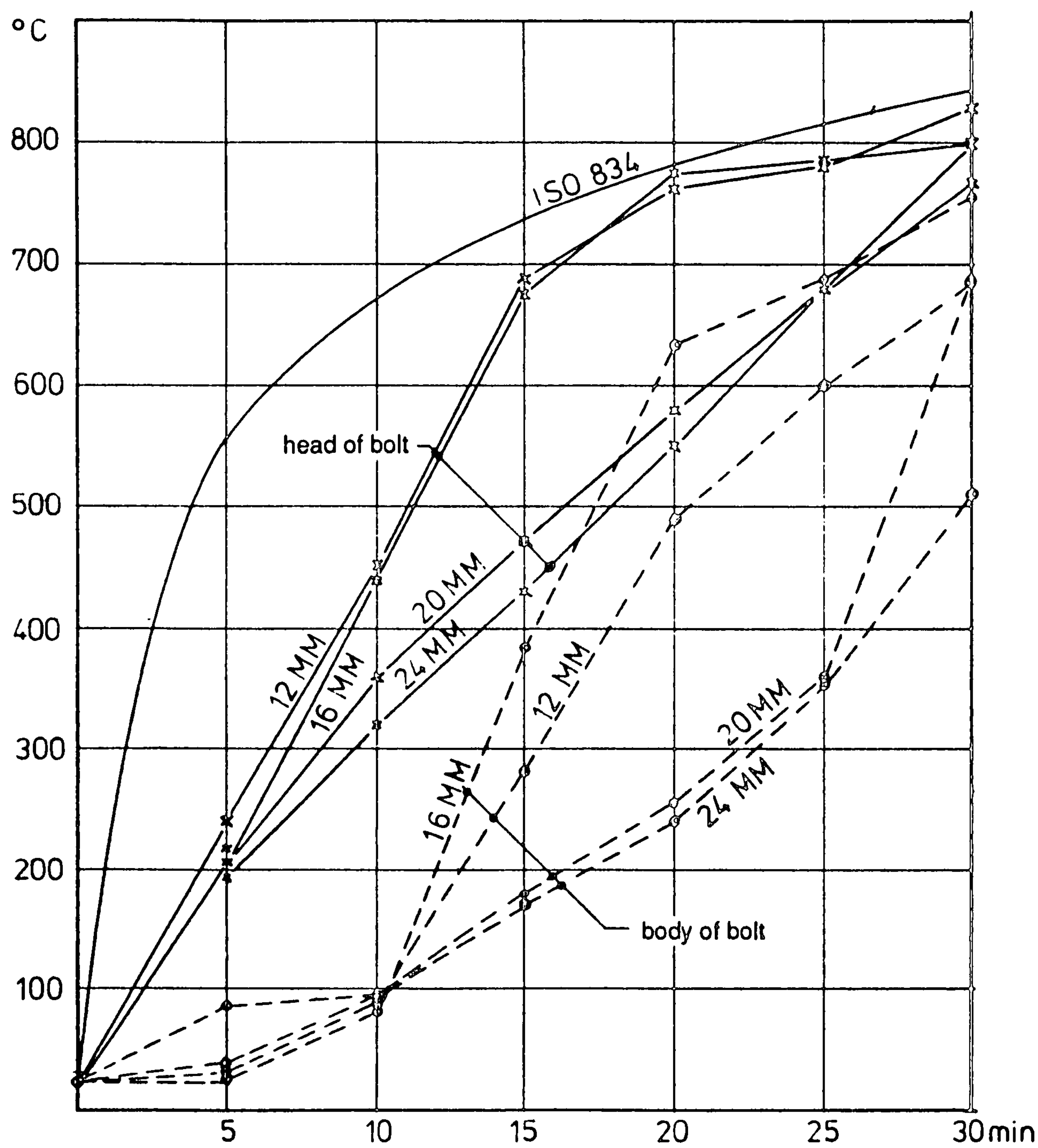


Figure 4.19: Temperature development in bolts of different diameters [4.1] [4.2].

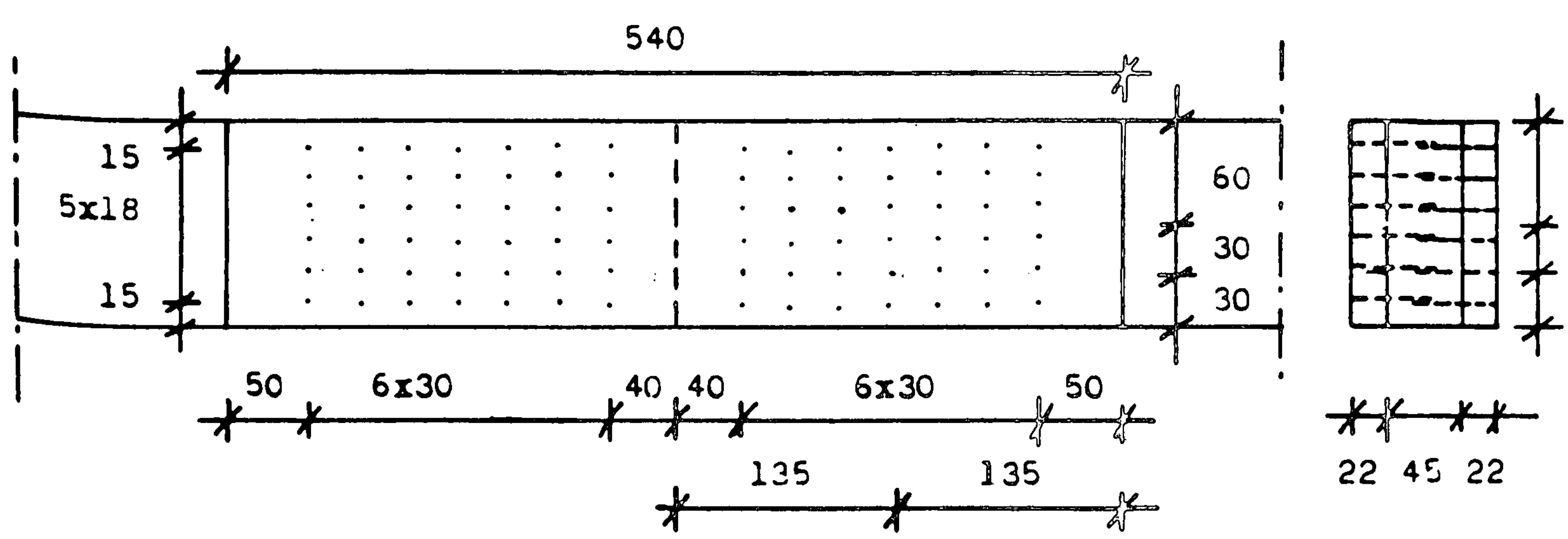


Figure 4.20: Fire tested timber joint with nailed timber gussets [4.13].

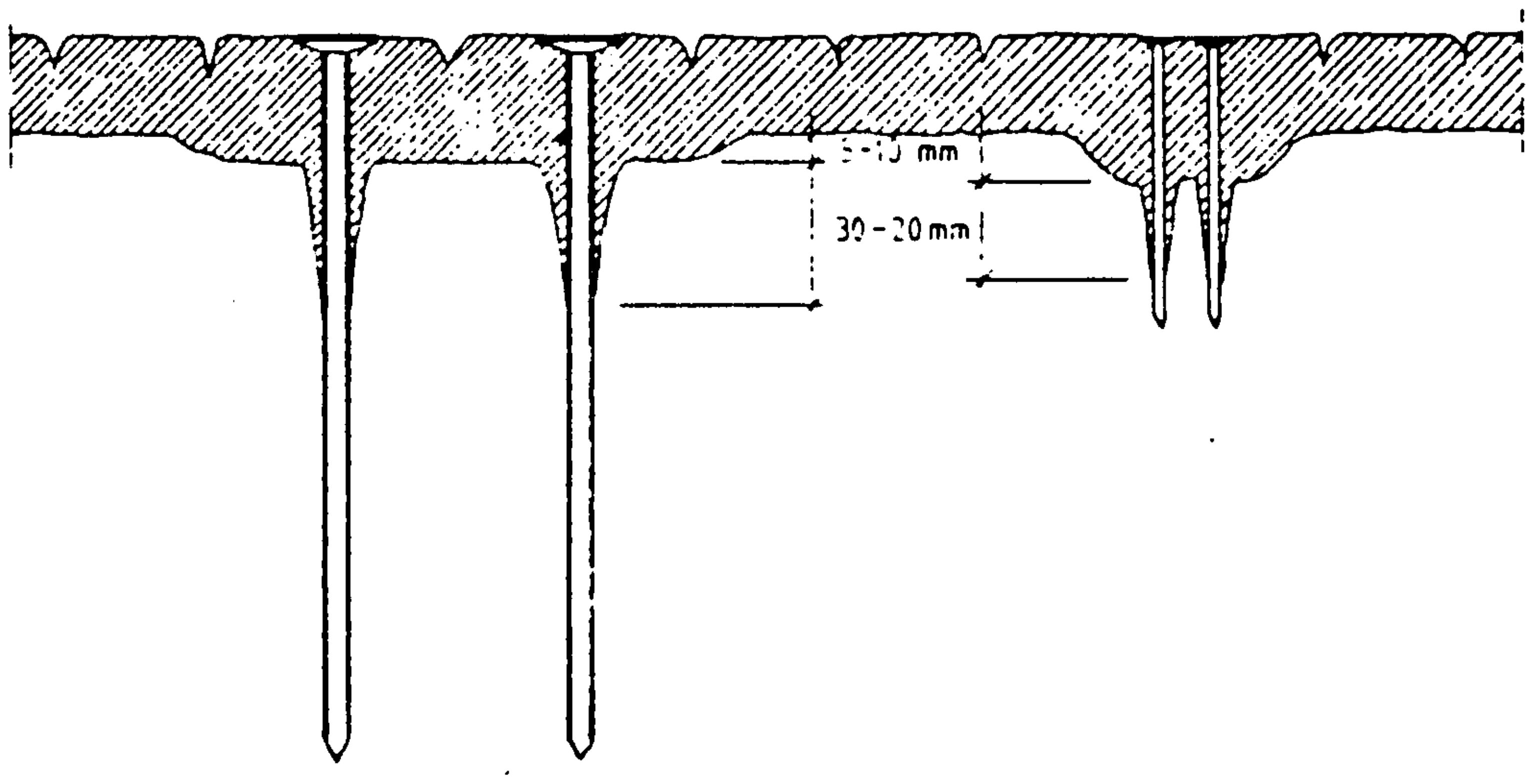


Figure 4.21: Schematic diagram of the penetration of the charred zone in the vicinity of large and small nails respectively [4.1], [4.2].

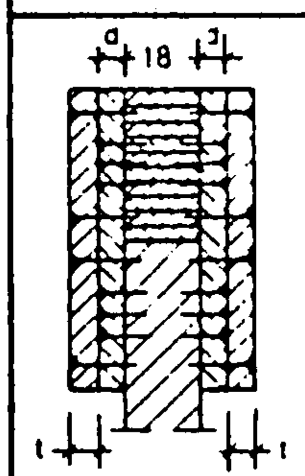
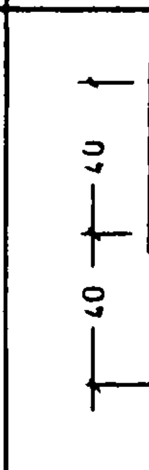
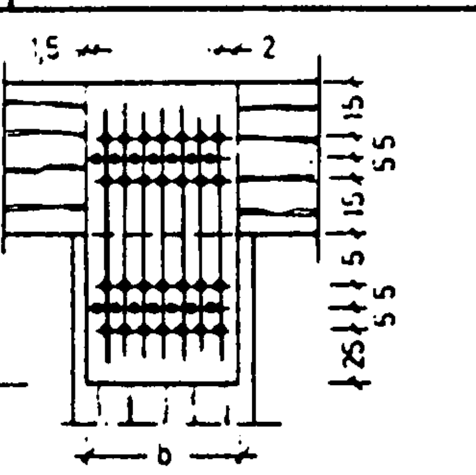
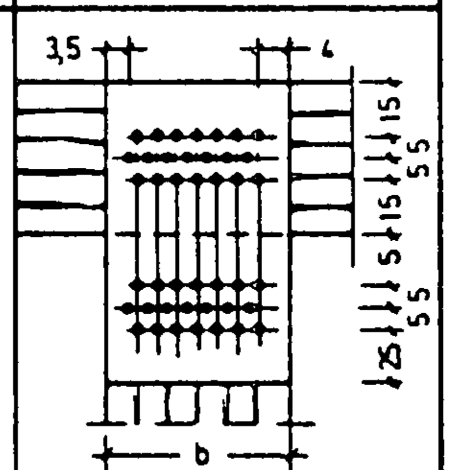
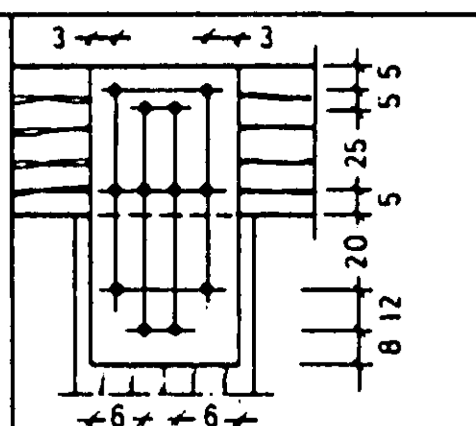
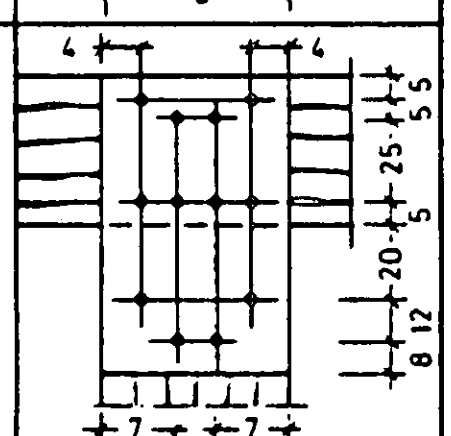
test	12	13	14
			
nailed cover plate test: 31/65 test 14: 34/90	–		
d / b	4 / 14	2,4 / 14 , predrilled	2,4 / 18 , predrilled:
t	–	2,4	4
FR (min)	22	48	68

Figure 4.22: Fire tested beam-to-column or truss joints, with nailed timber gussets [4.14].

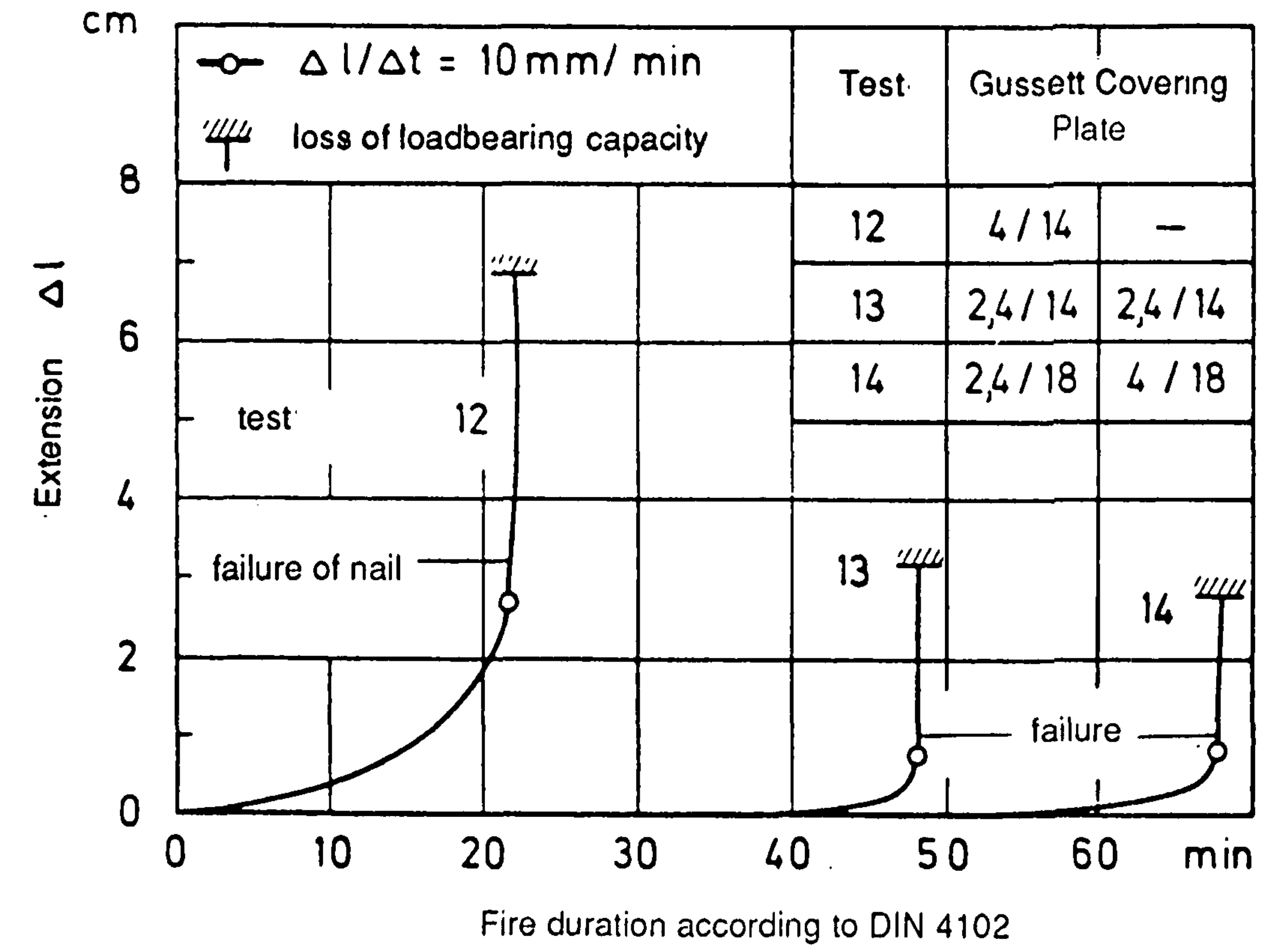


Figure 4.23: Deformations of joints shown in Figure 4.22 in the fire tests [4.14].

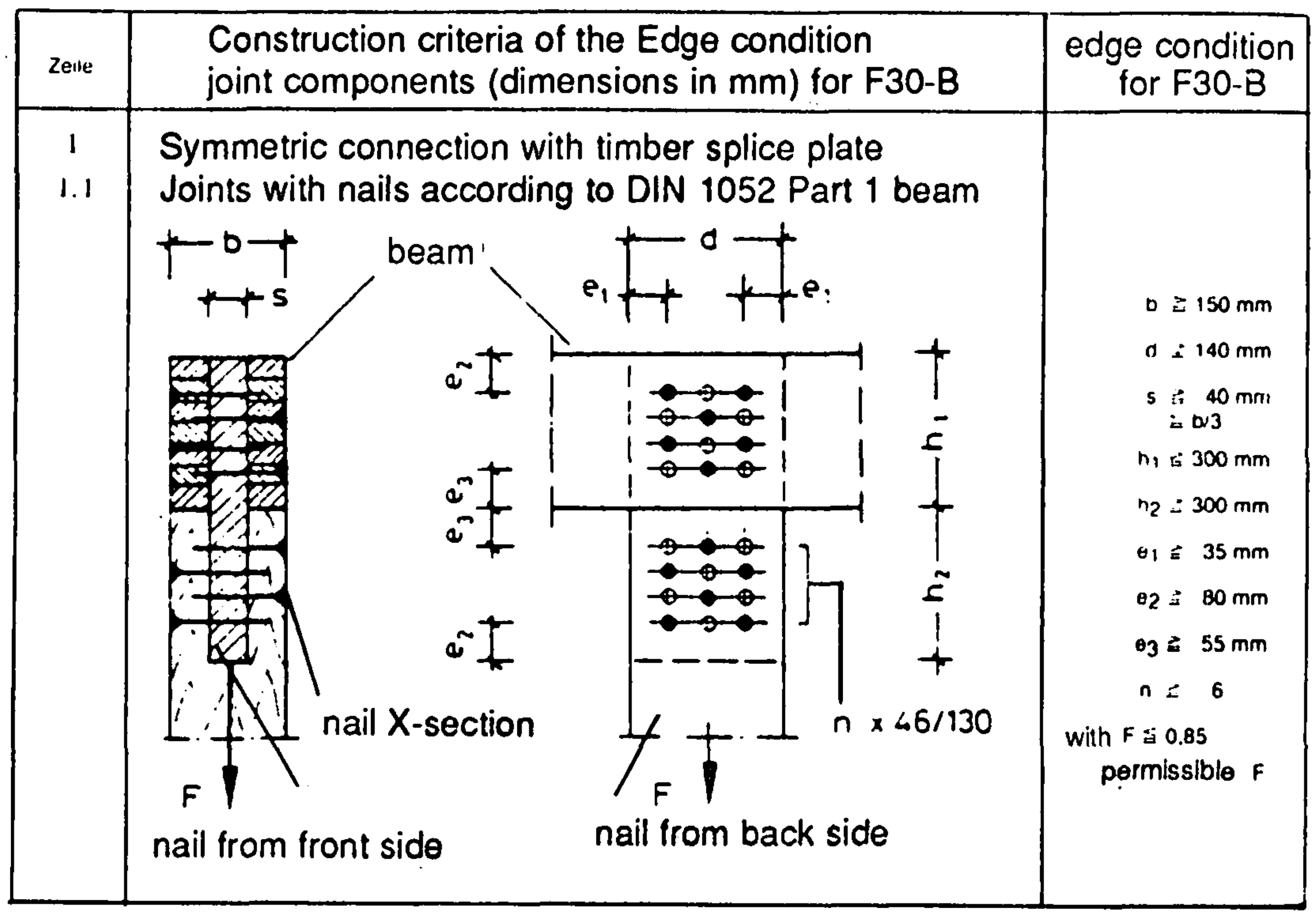
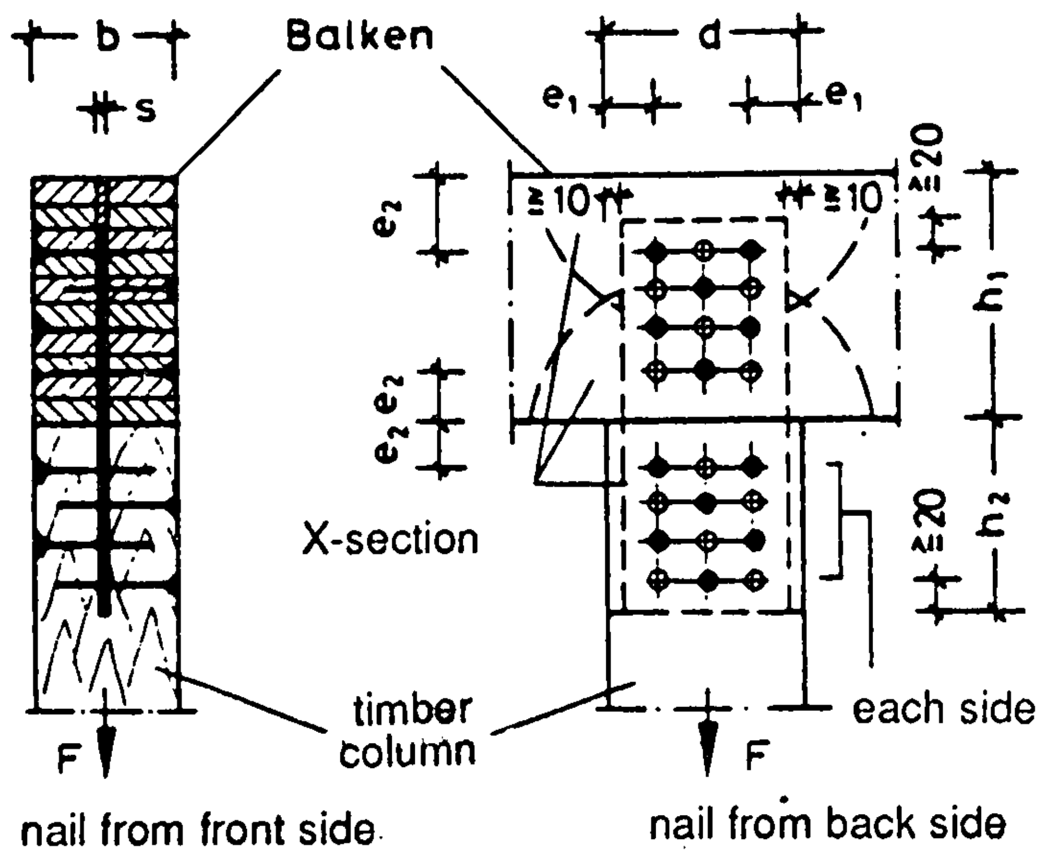


Figure 4.24: Fire tested beam-to-column joints in framework with timber spliced-in plate and double sided nailing [4.14], [4..16].

<table>
<tr><th>line</th><th>Construction criteria of the Edge condition joint components (dimensions in mm) for F30-B</th><th>Edge condition 'F 30-B</th></tr>
<tr><td>3</td><td>Symmetric connection with steel splice plate according to DIN 17 162 Part 1</td><td></td></tr>
<tr><td>3.1</td><td>joints with nails (predrilled)



</td><td>$b \geq 150$ mm
$d \geq 140$ mm
$s = 2$ mm
$h_1 \geq 300$ mm
$h_2 \geq 260$ mm
$e_1 \geq 40$ mm
$e_2 \geq 60$ mm
$n \geq 6$</td></tr>
</table>

Figure 4.25: Fire tested beam-to-column joints in framework with steel spliced-in plate and double sided nailing [4.14], [4.16].

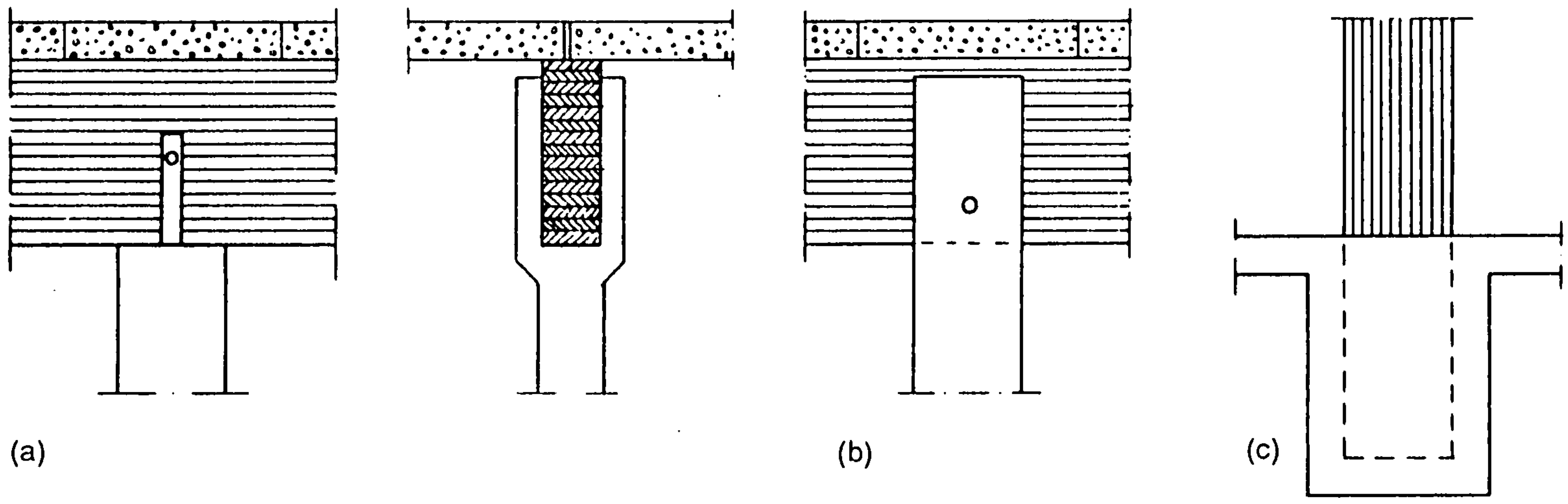


Figure 5.1: Examples of joints between concrete and glulam timber construction.

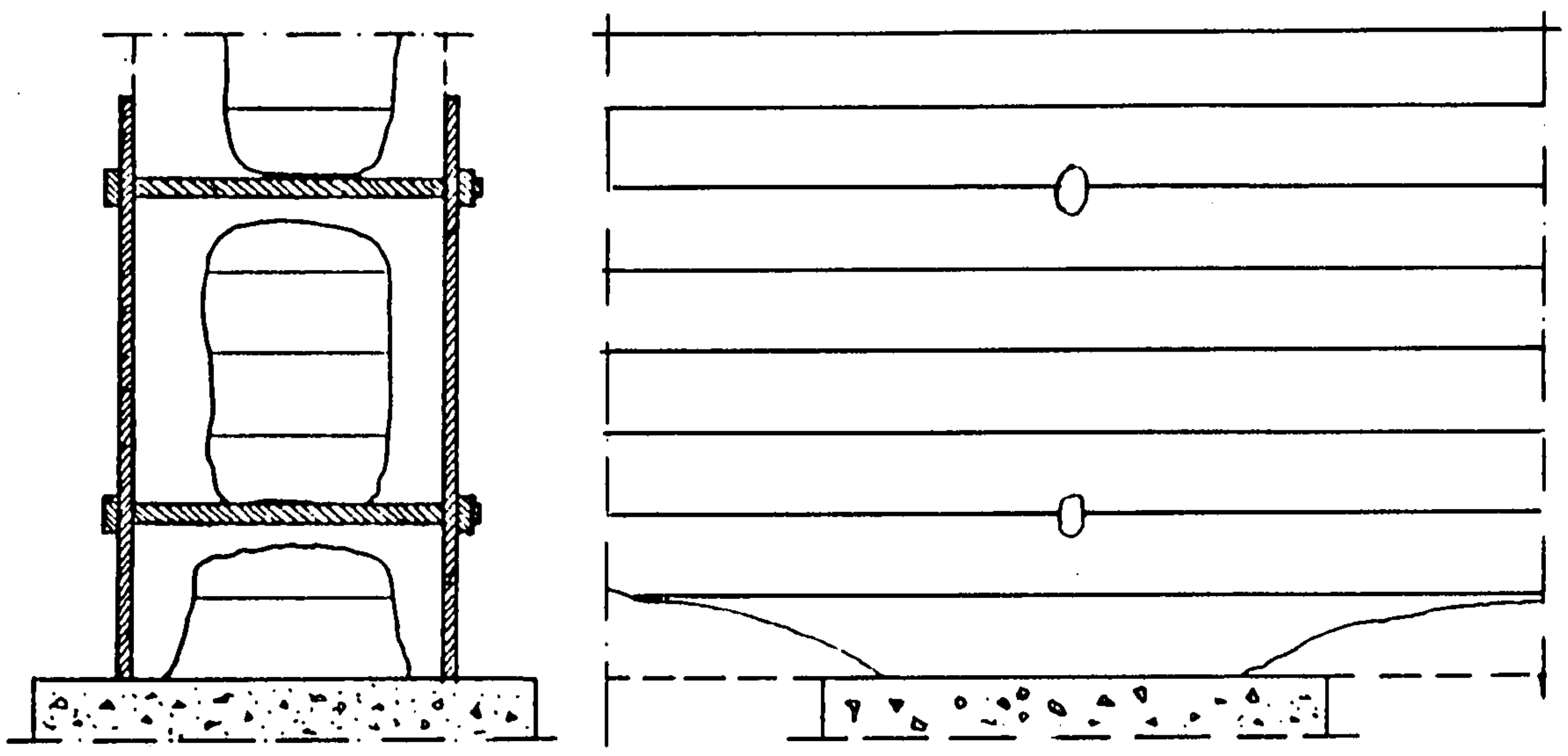


Figure 5.2: Fire tested beam-to-column joint in Figure 5.1a. Vertical sections across and along the beam after 60 minutes of standard fire.

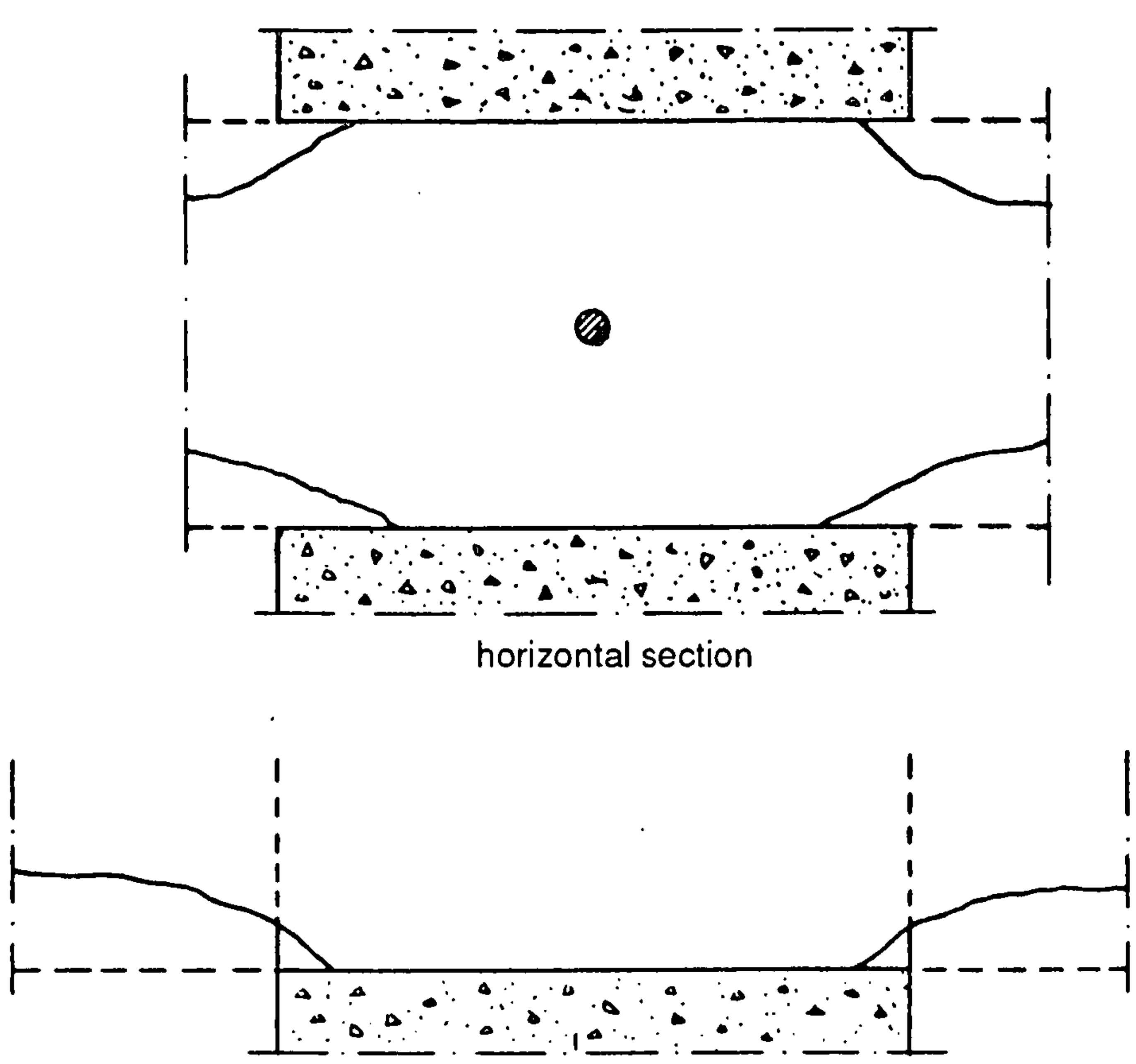


Figure 5.3: Fire tested beam-to-column joint in Figure 5.1b. Horizontal and vertical sections along the beam after 60 minutes of standard fire.

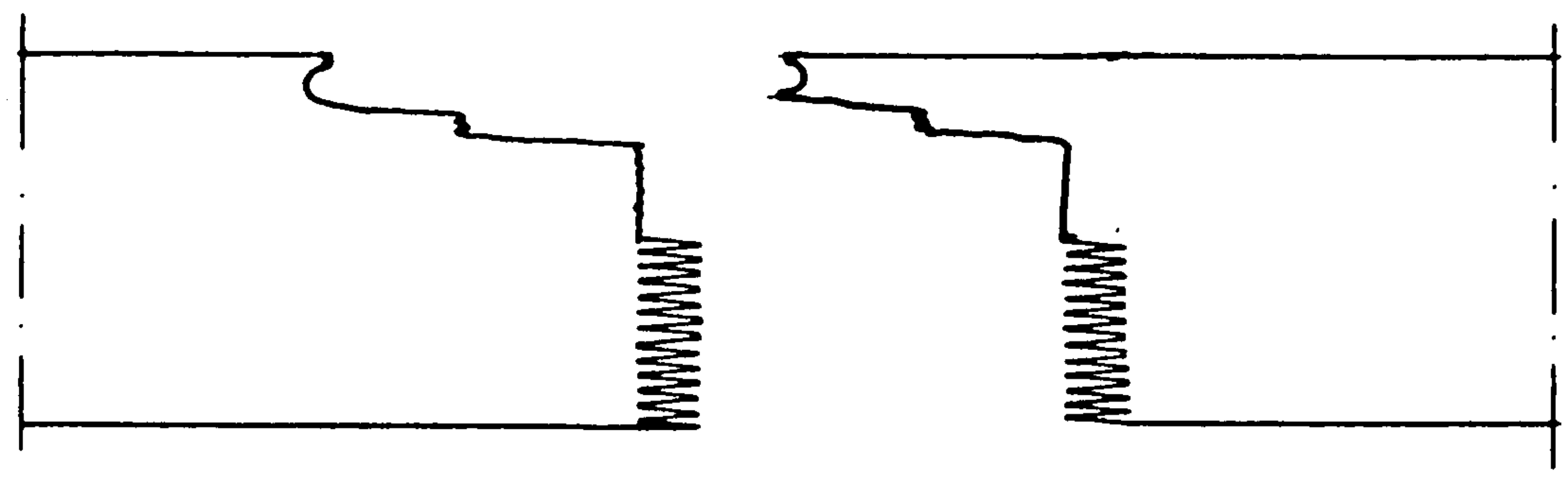


Figure 6.1: Typical failure of a finger joint which was a combination of fracture at the base of the fingers, the side of the fingers, adjoining knot or other natural defects in the timber.

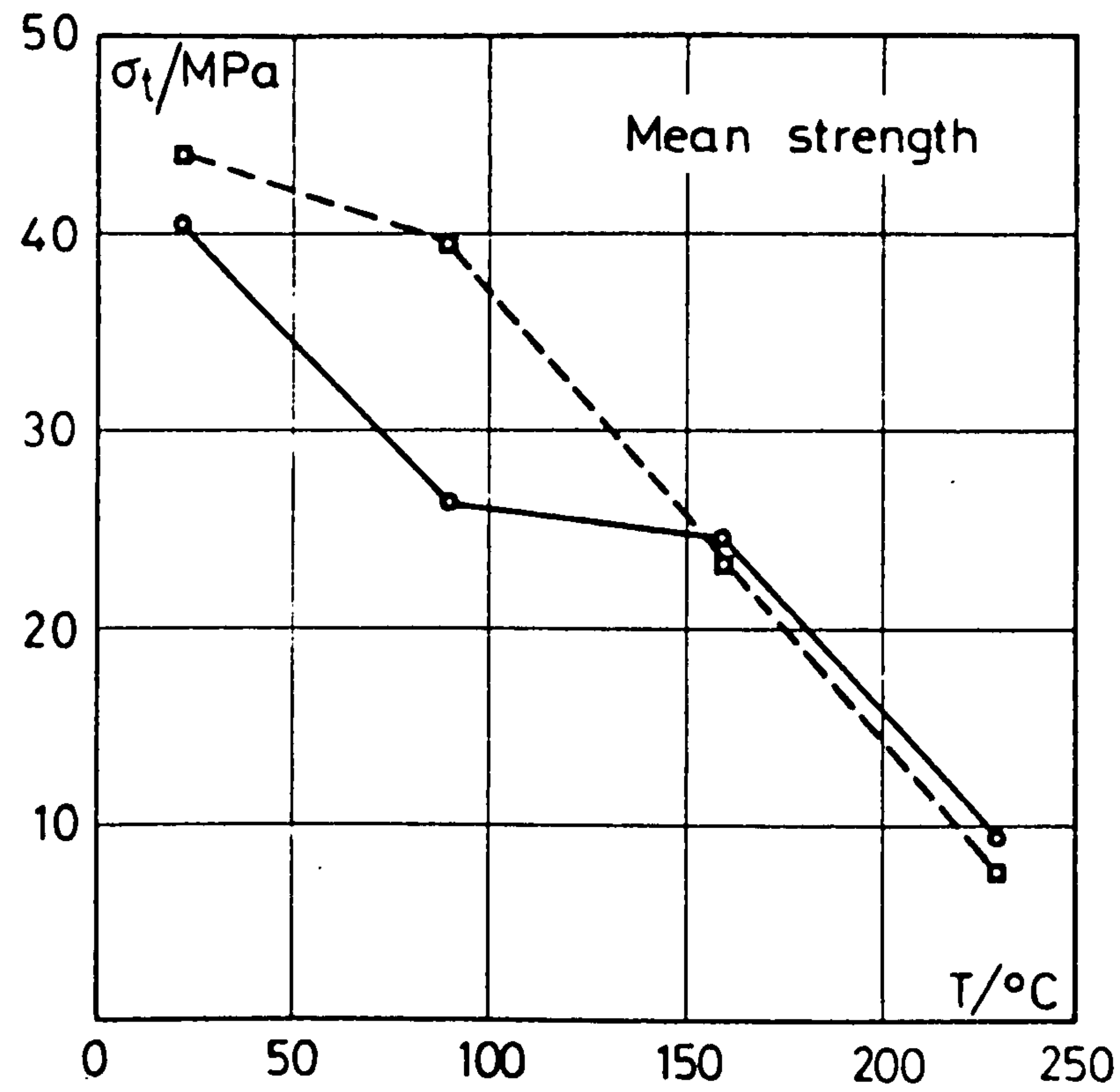


Figure 6.2: Experimental strength of timber with (□ ___) and without (o ____) finger joints at different temperatures [6.9].

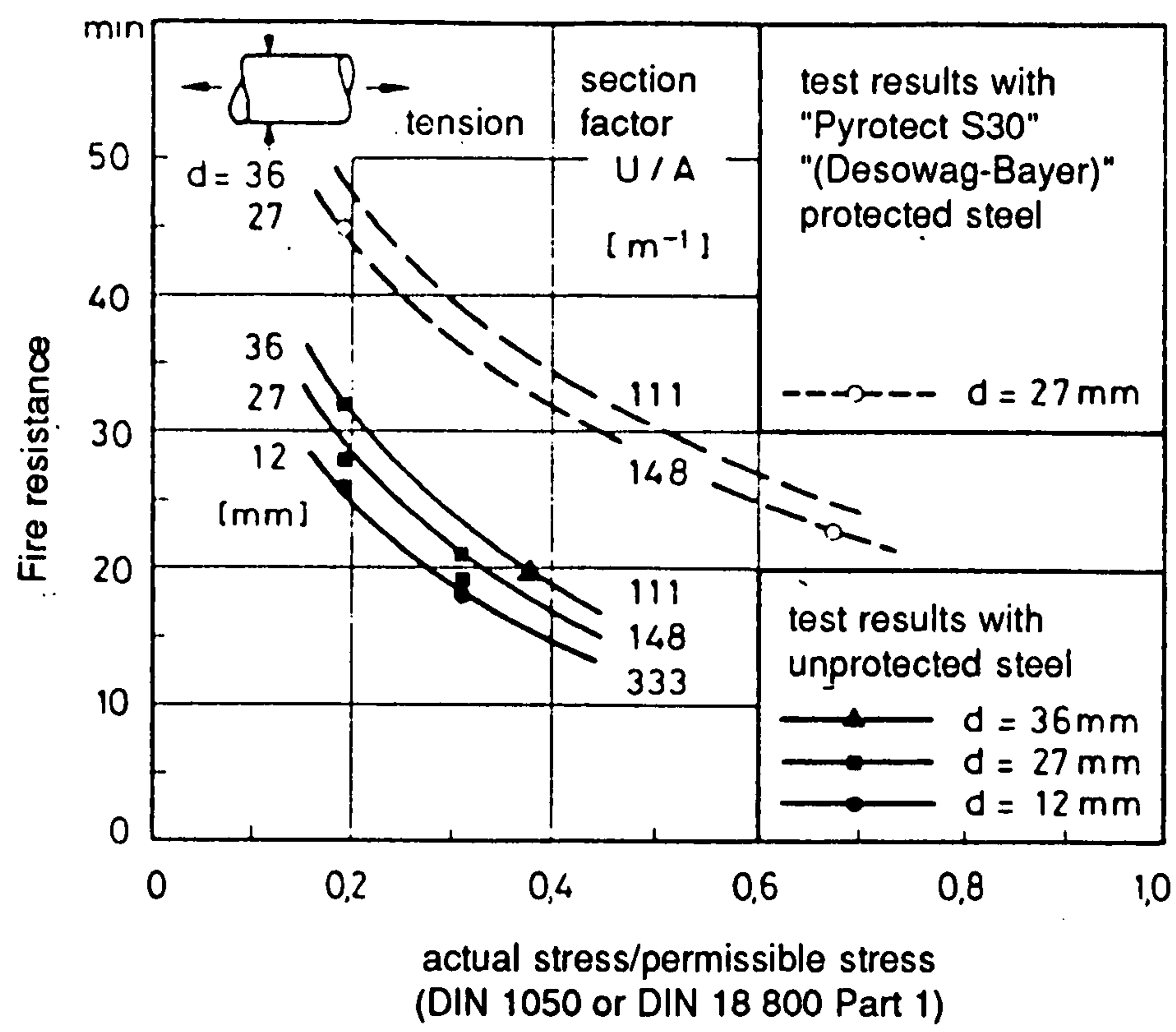


Figure 7.1: The fire resistance of round steel bars with and without fire resistant coat ings as a function of the permissible stress and the bar diameter [7.5].

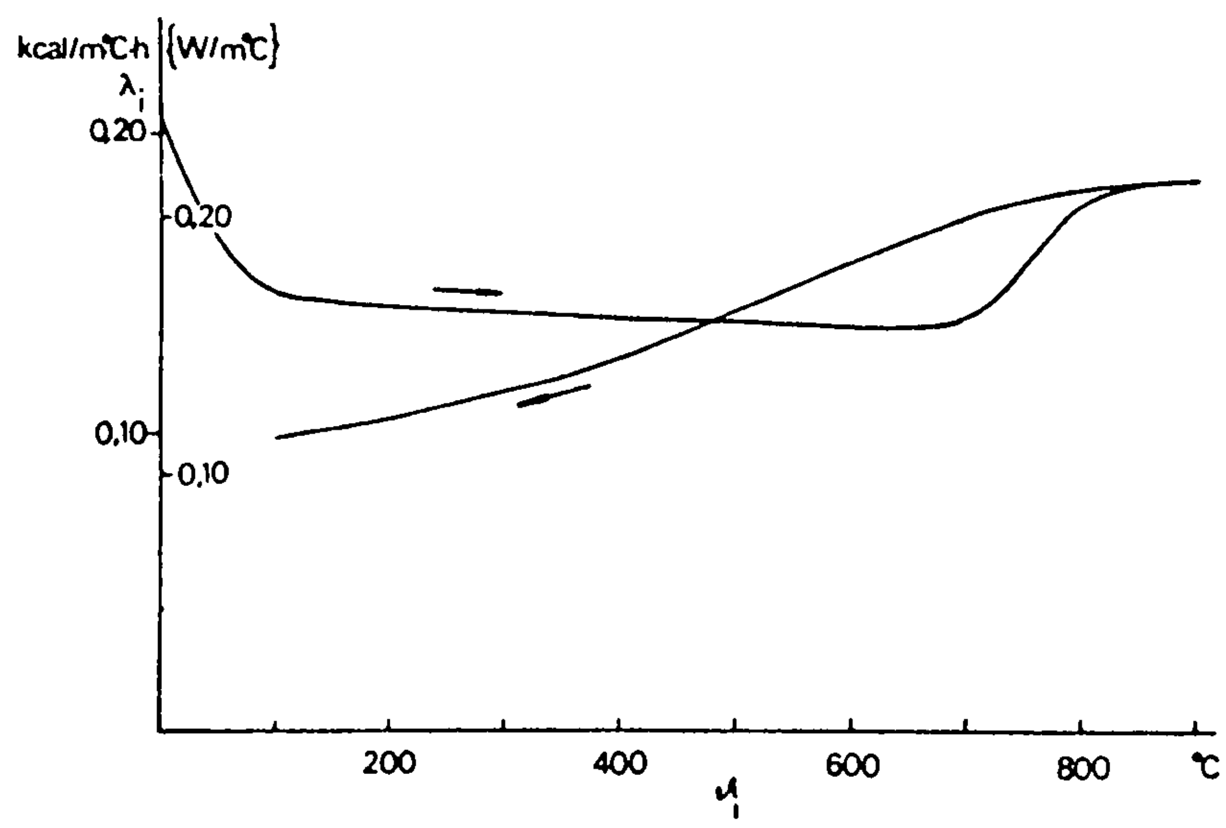


Figure 7.2: Conductivity of fire resistant plaster of the "Jimotherm' type as a function of the temperature in the plaster

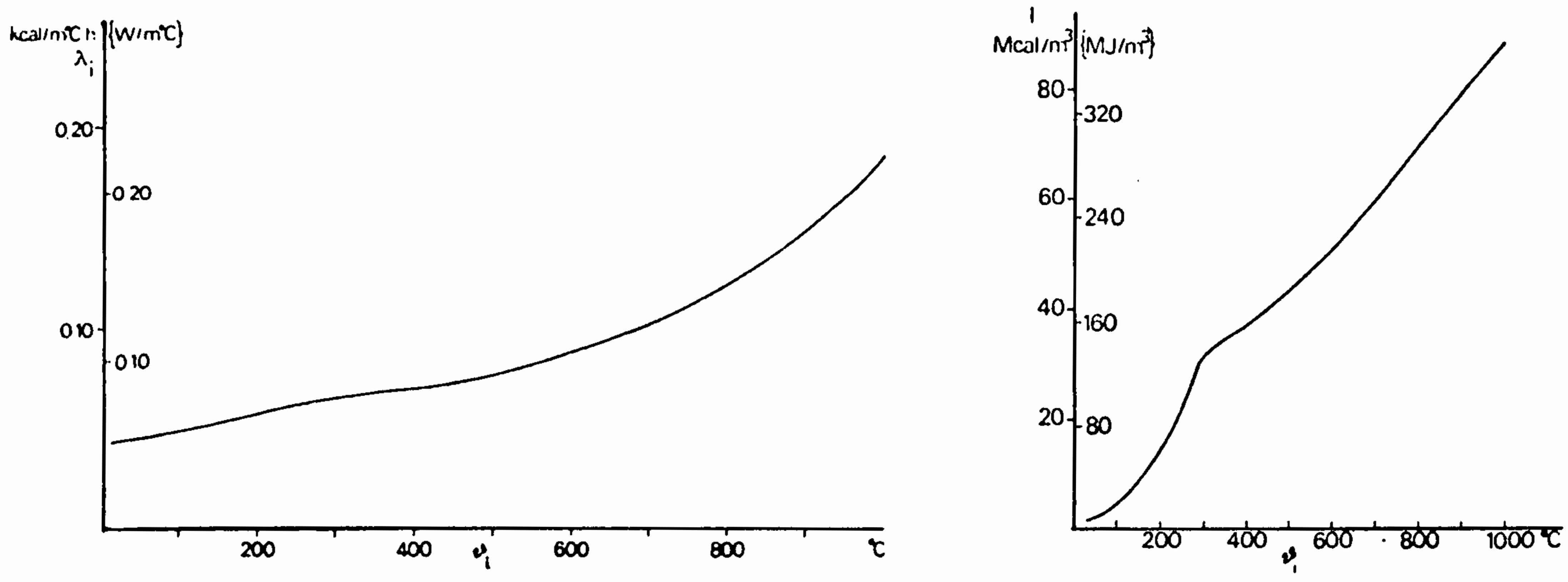


Figure 7.3: Conductivity and heat absorption characteristics of sprayed mineral wool of the type "Pyroguard 101" (T= 250 kg/m³) as a function of the temperature in the insulation layer.

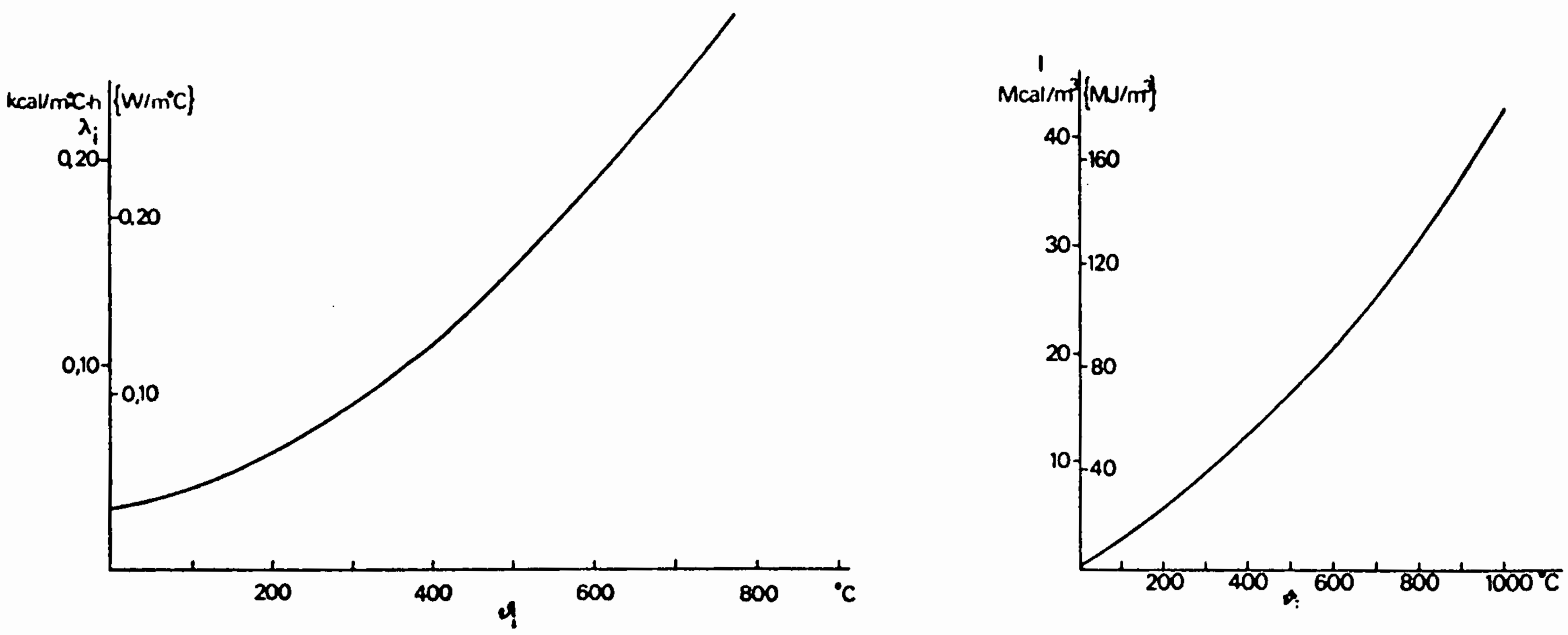


Figure 7.4: Conductivity and heat absorption characteristics in mineral wool boards (T= 150 kg/m³) as a function of the material temperature [7.1].

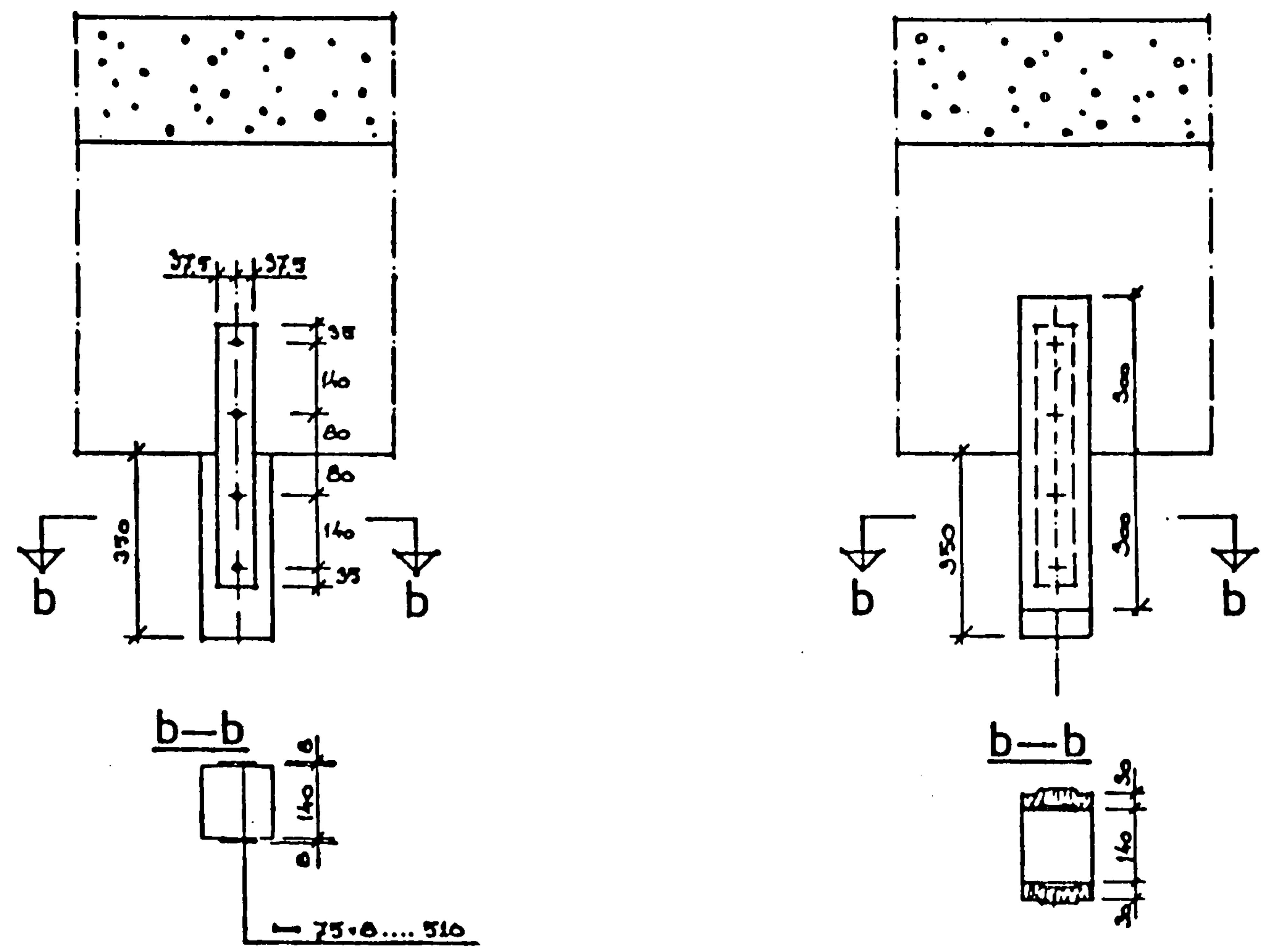


Figure 7.5: Fire tested beam-to-column joints of steel gussets and screws with or without mineral wool insulation.

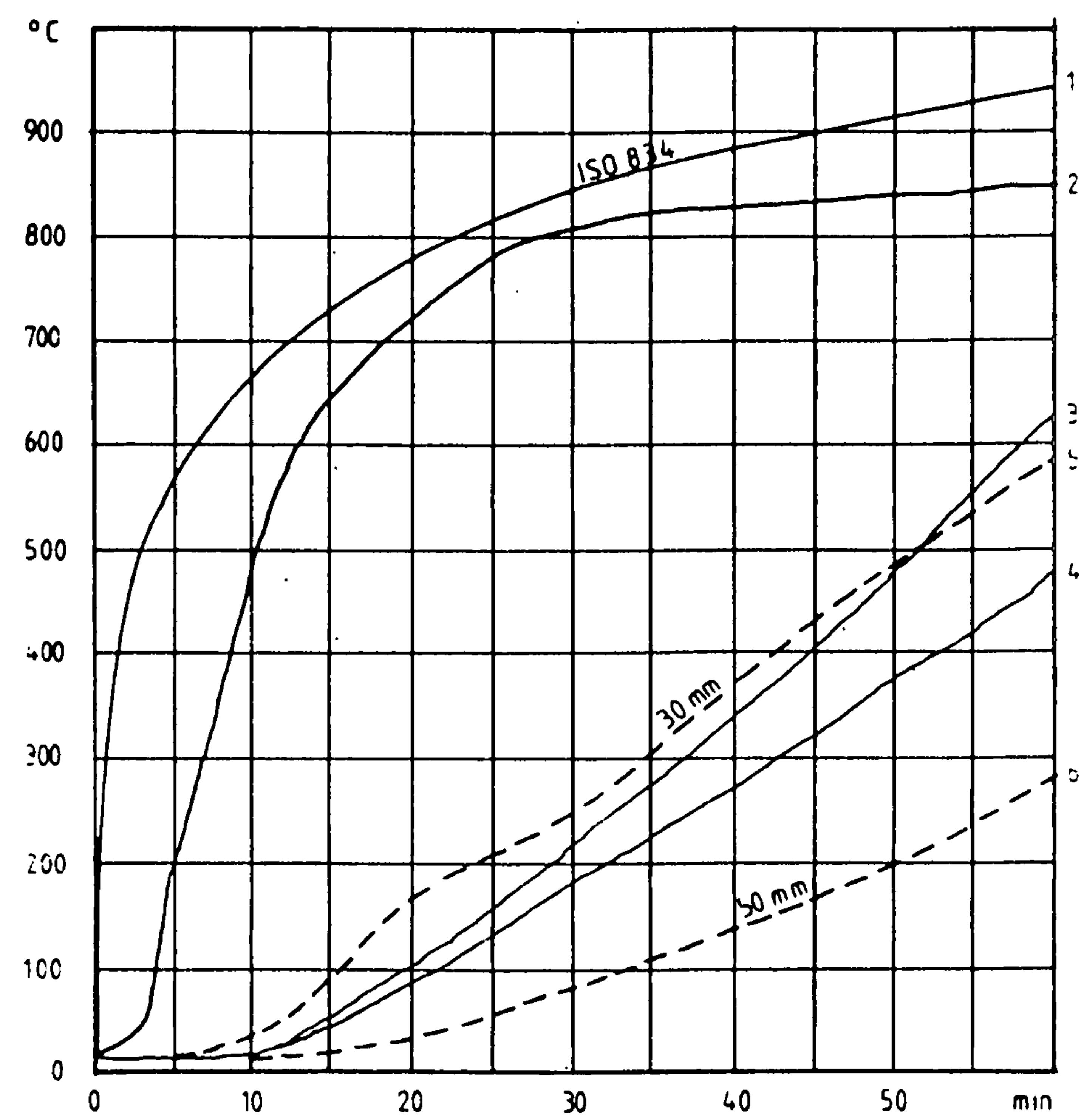


Figure 7.6: Temperature development in steel gusset without (curve 2) and with (curves 3 and 4) the mineral wool insulation as shown in Figure 7.5 [7.9]. Similar characteristics with mineral wool insulation are shown in Figure 7.7 (curves 5 and 6).

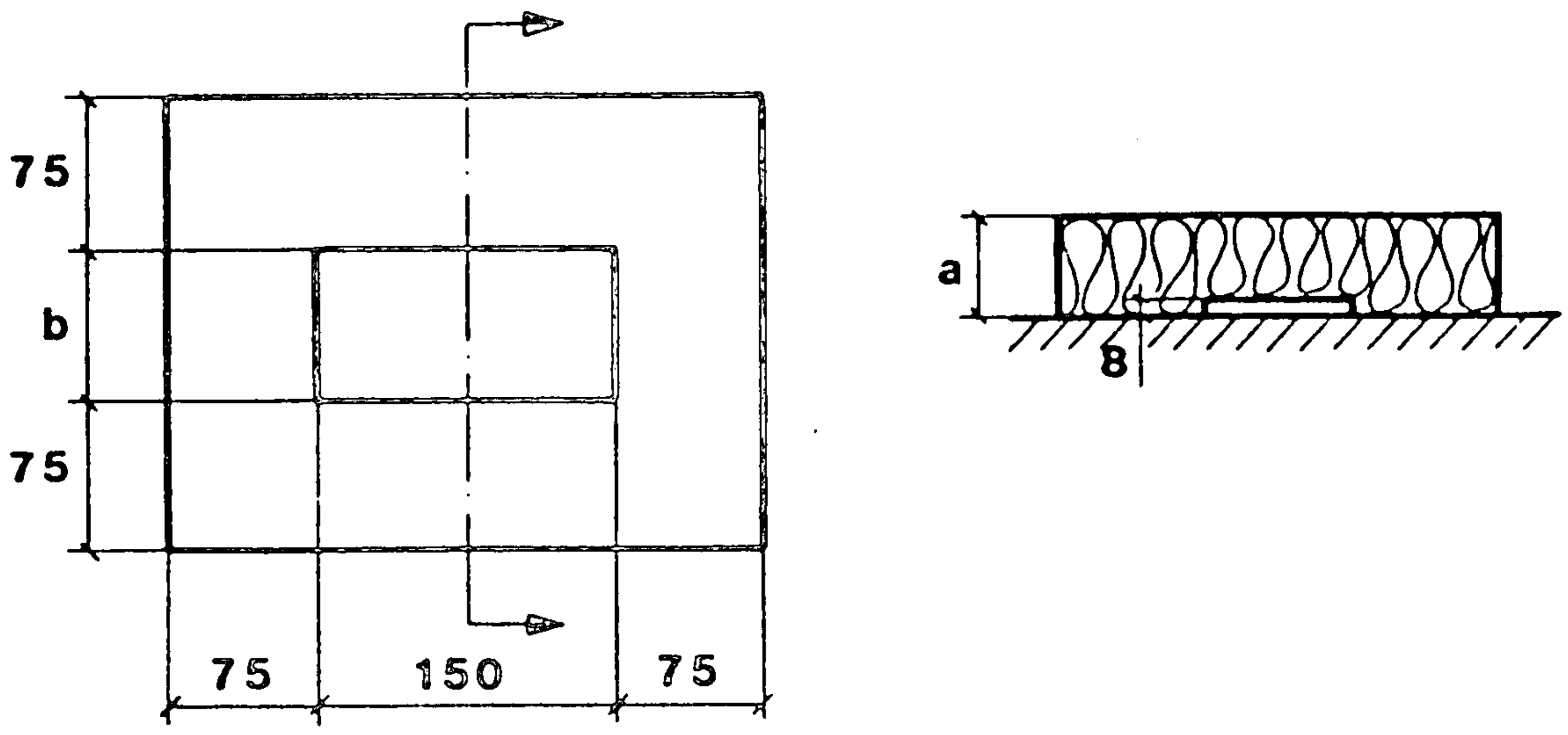


Figure 7.7: Test samples for studying the temperature development during fire in flat steel plate insulated with mineral wool [7.8] where a = 30 or 50 mm and b = 75 or 125 mm.

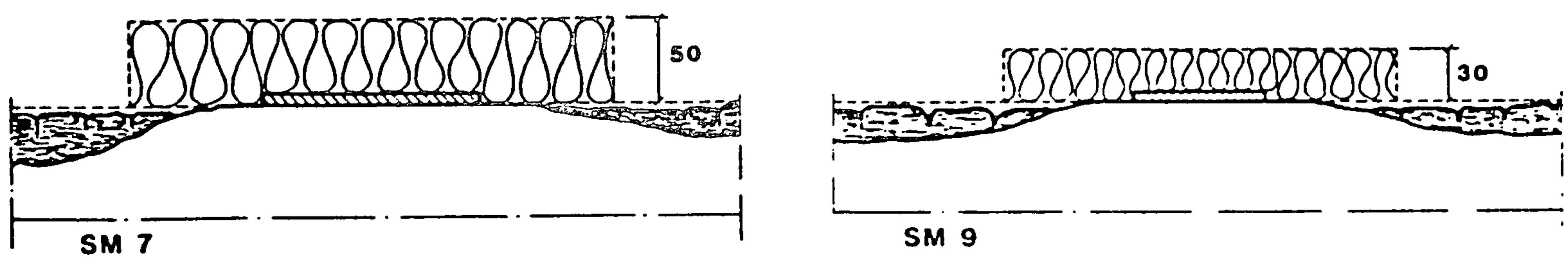


Figure 7.8: Sections through the mineral wool insulated plates as shown in Figure 7.7 after 60 minutes of standard fire [7.8].

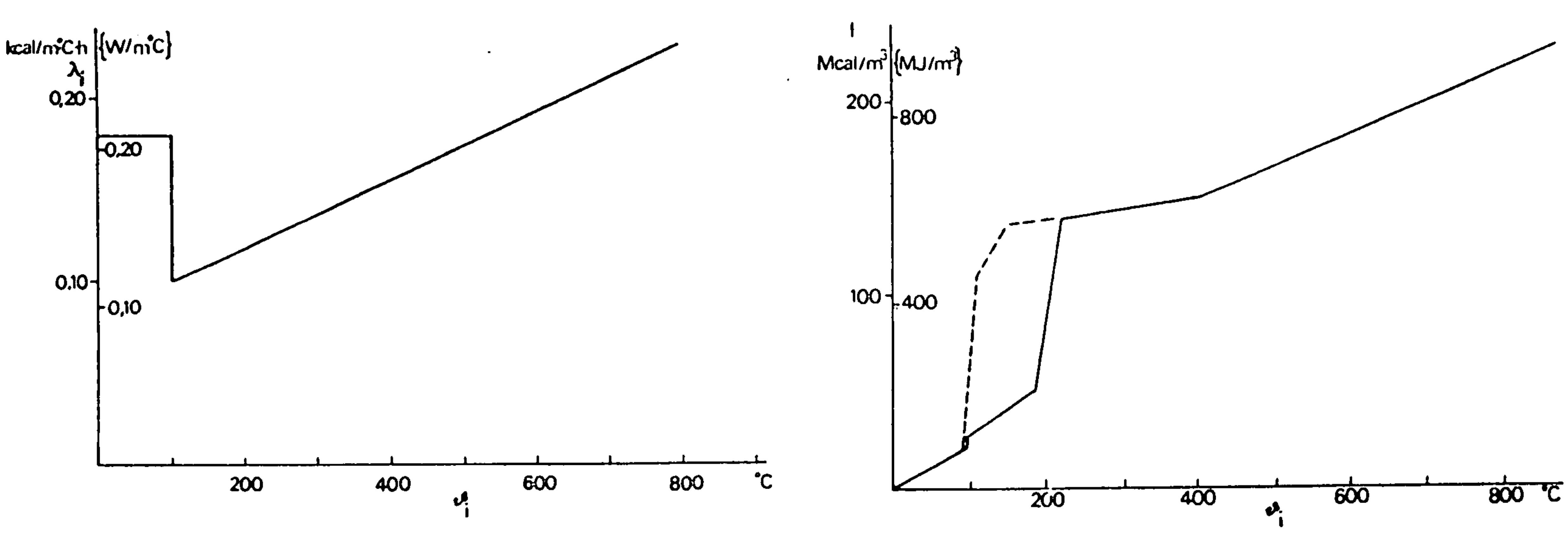
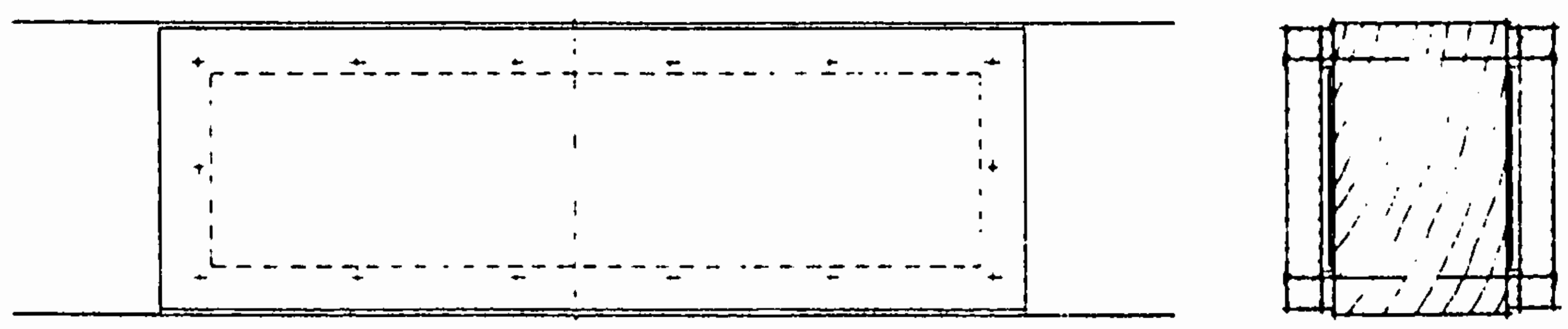


Figure 7.9: Conductivity and heat absorption characteristics of gypsum plasterboard (= 790 kg/m³) as a function of the temperature in the board [7.1]. The full curve is for the board exposed to direct fire and the dotted curve is for the board which was not exposed directly to the fire.

Test CO1 - CO3
Cover Plate: 450 x 145 x 22 mm plywood, fastened with 14 nos. 40/60 nails
Side packing: 20 x 6 mm plywood
Test CO4
Cover Plate: 450 x 145 x 18.4 mm plywood impregnated with fire retardant, fastened with 14 nos. 40/60 nails.
Side packing: 20 x 6 mm plywood



Test CO5
Cover plate: Internal plate dimensions 400 x 100 x 9 mm plywood impregnated with fire retardent, fixed with 8 nos. 20/40 bright nails through the plate. External plate dimensions 450 x 145 x 9 mm plywood impregnated with fire retardant and fastened with 14 nos. 40/60 nails.
Side Packing: 20 x 14 mm plywood

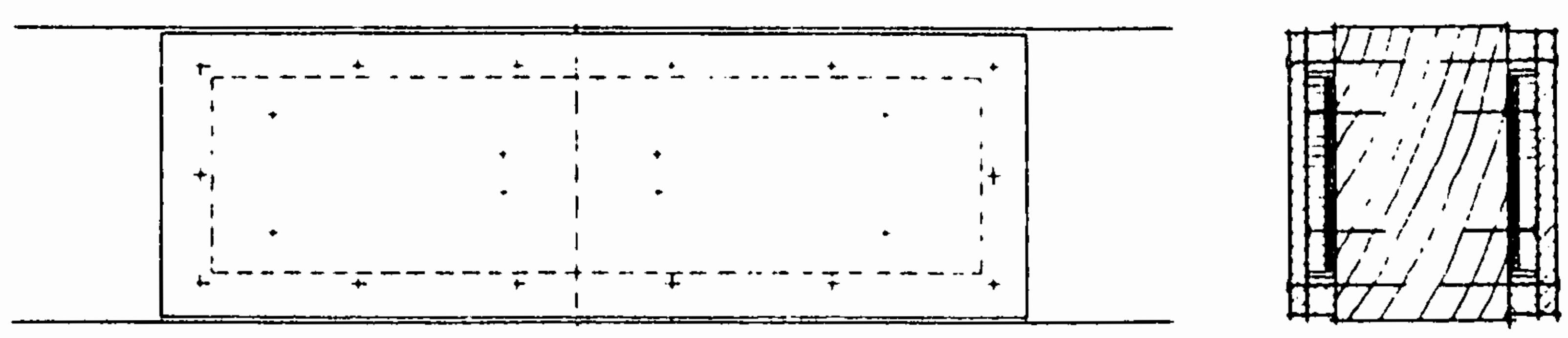


Figure 7.10: Fire tested timber joints with plywood protected nail-on gussets [7.10].

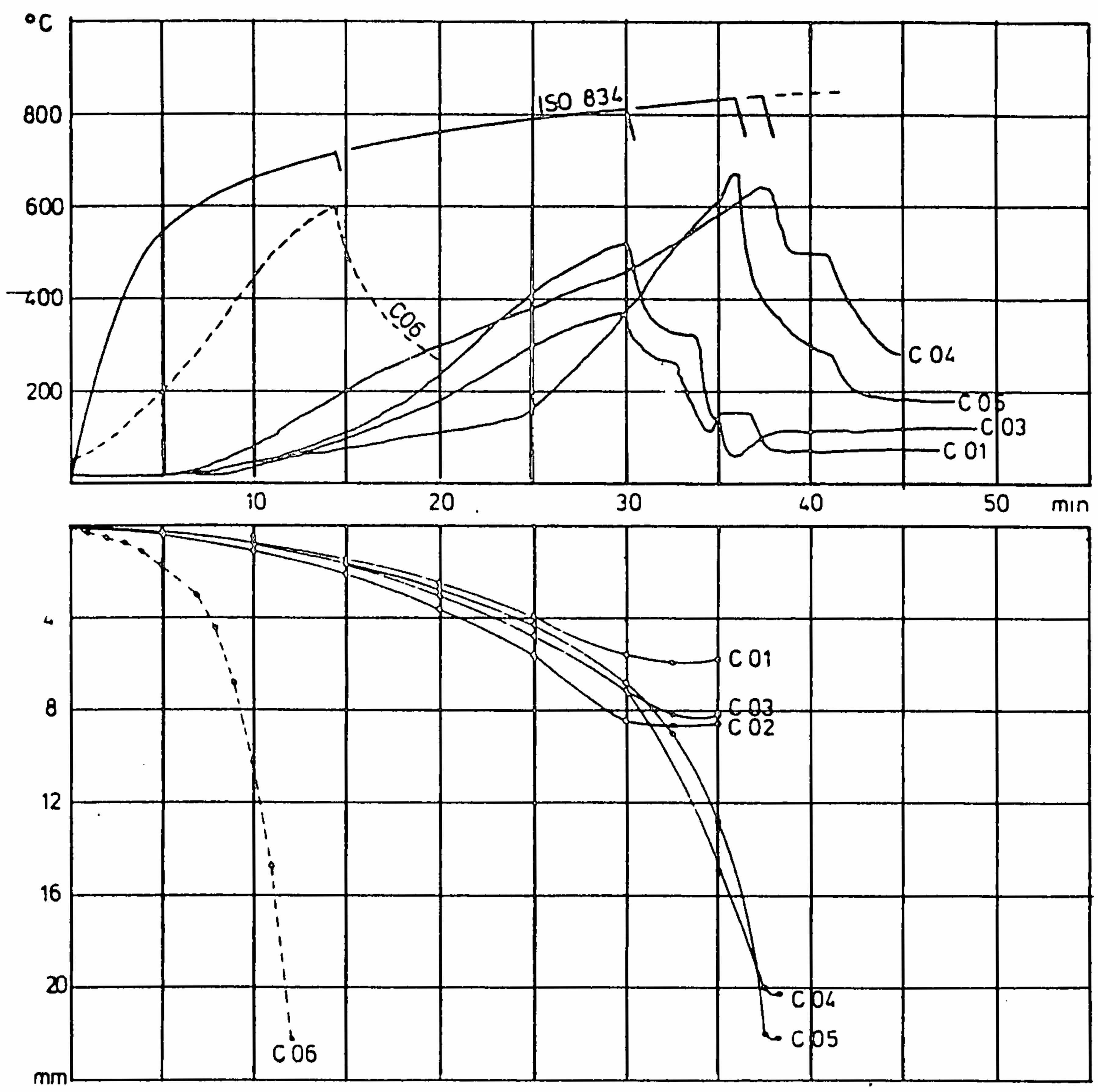


Figure 7.11: The temperature and deformation characteristics of the fire tested timber joint with plywood protected nail-on gussets as shown in Figure 7.10 [7.10].

Test number	Load	§	Δ§
CO1	16,3 kN	5,5 mm	0,9 mm
CO2	16,3 kN	8,3 mm	1,0 mm
CO3	16,3 kN	7,1 mm	1,0 mm

Table 7.5: § = elongation of the joint after 30 minutes of standard fire condition in accor - dance with ISO 834-1975. Δ§ = elongation of the joint when the load was increased from 16.3 kN to 25 kN, 24 hours after the fire test.

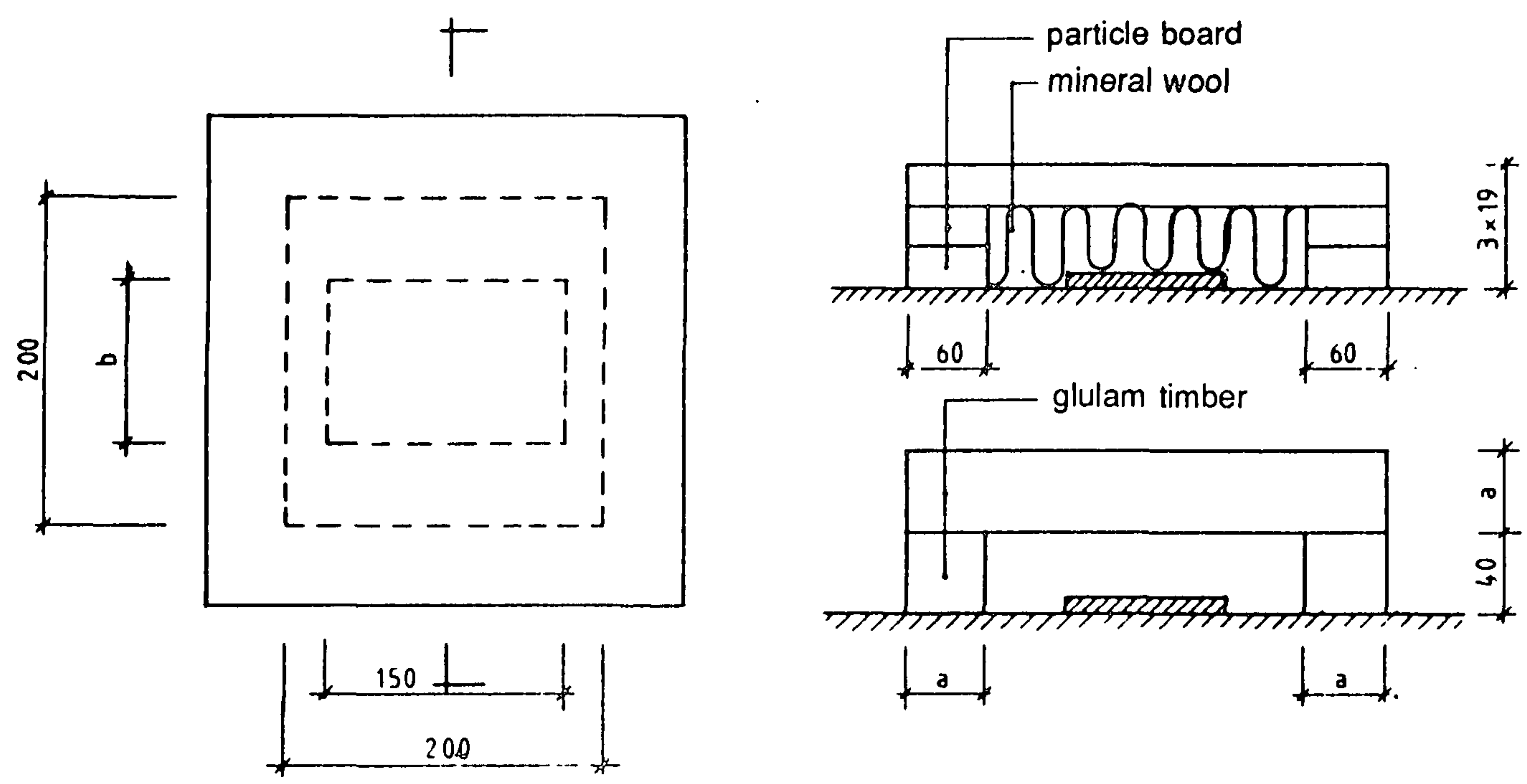


Figure 7.12: Samples for the study of temperature development during fire for steel gussets which have been protected with glulam timber or particle board [7.7].

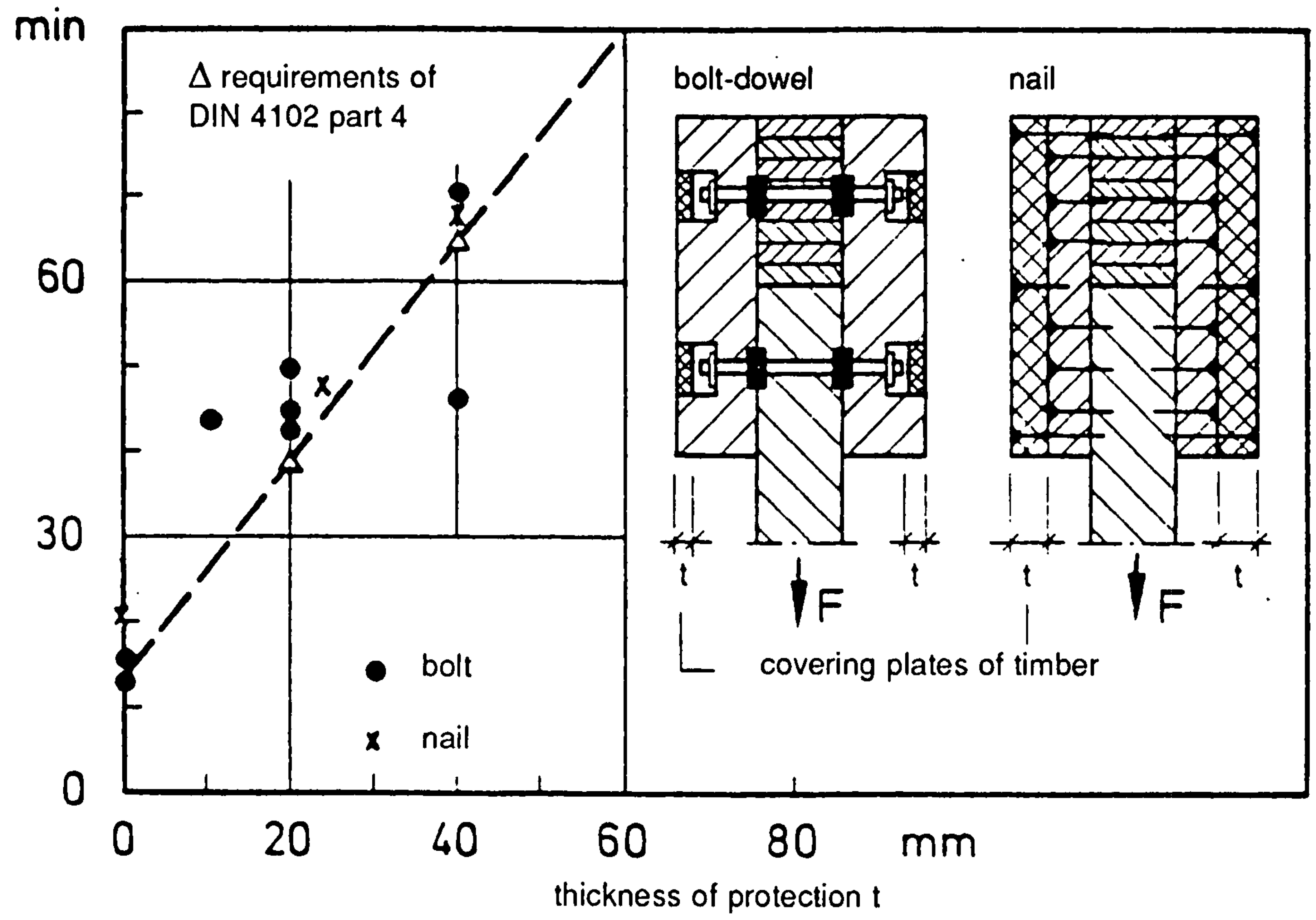
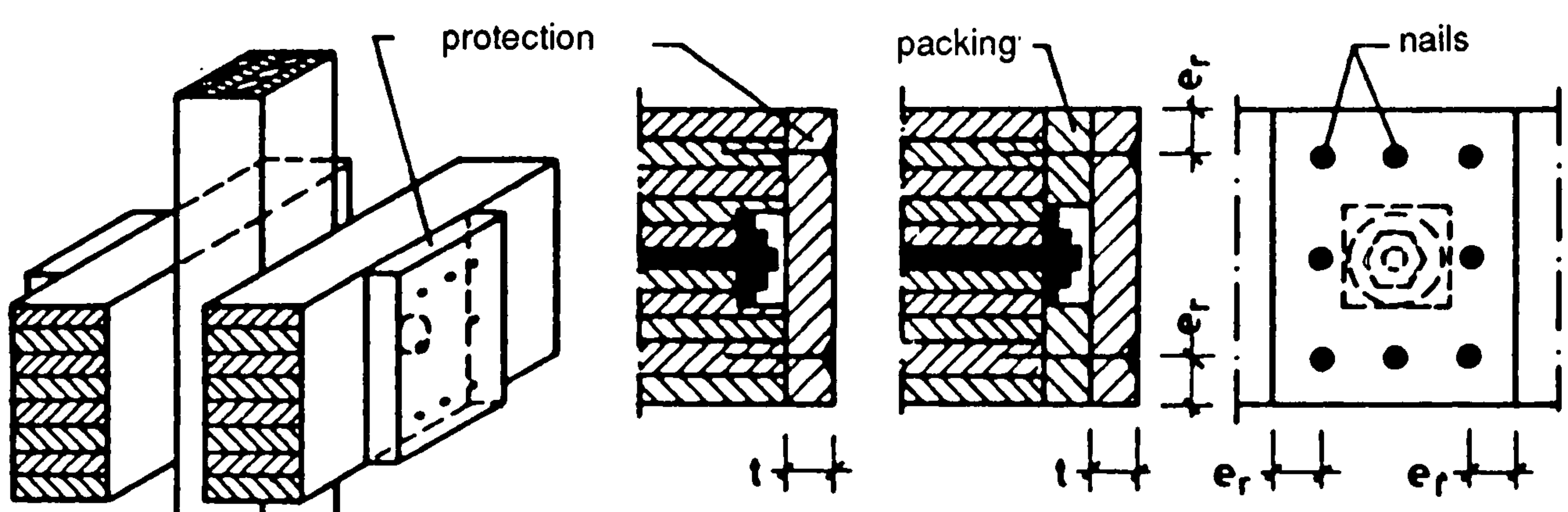


Figure 7.13: Experimental results of the relationship between fire resistance and thick ness of the wooden plug or timber protection [7.5].



	F 30	F 60
e_r [mm]	30	60
t (mm)	20	40

Figure 7.14: The required thicknesses of protections for bolted joints [7.5].

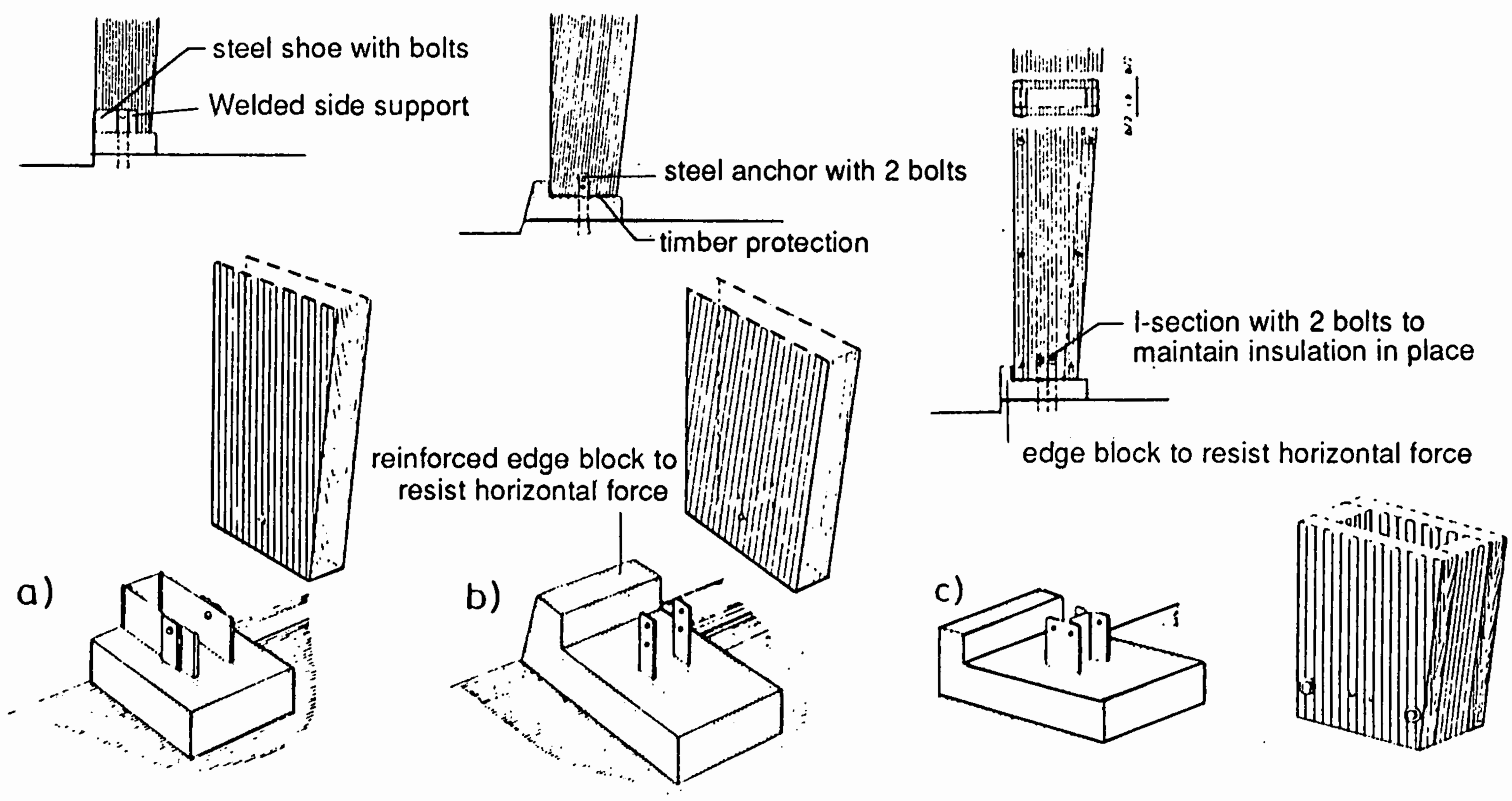


Figure 8.1: Hinged column bases, where the loads are transferred through bearing [8.1], present no problem even when high fire resistance is required.

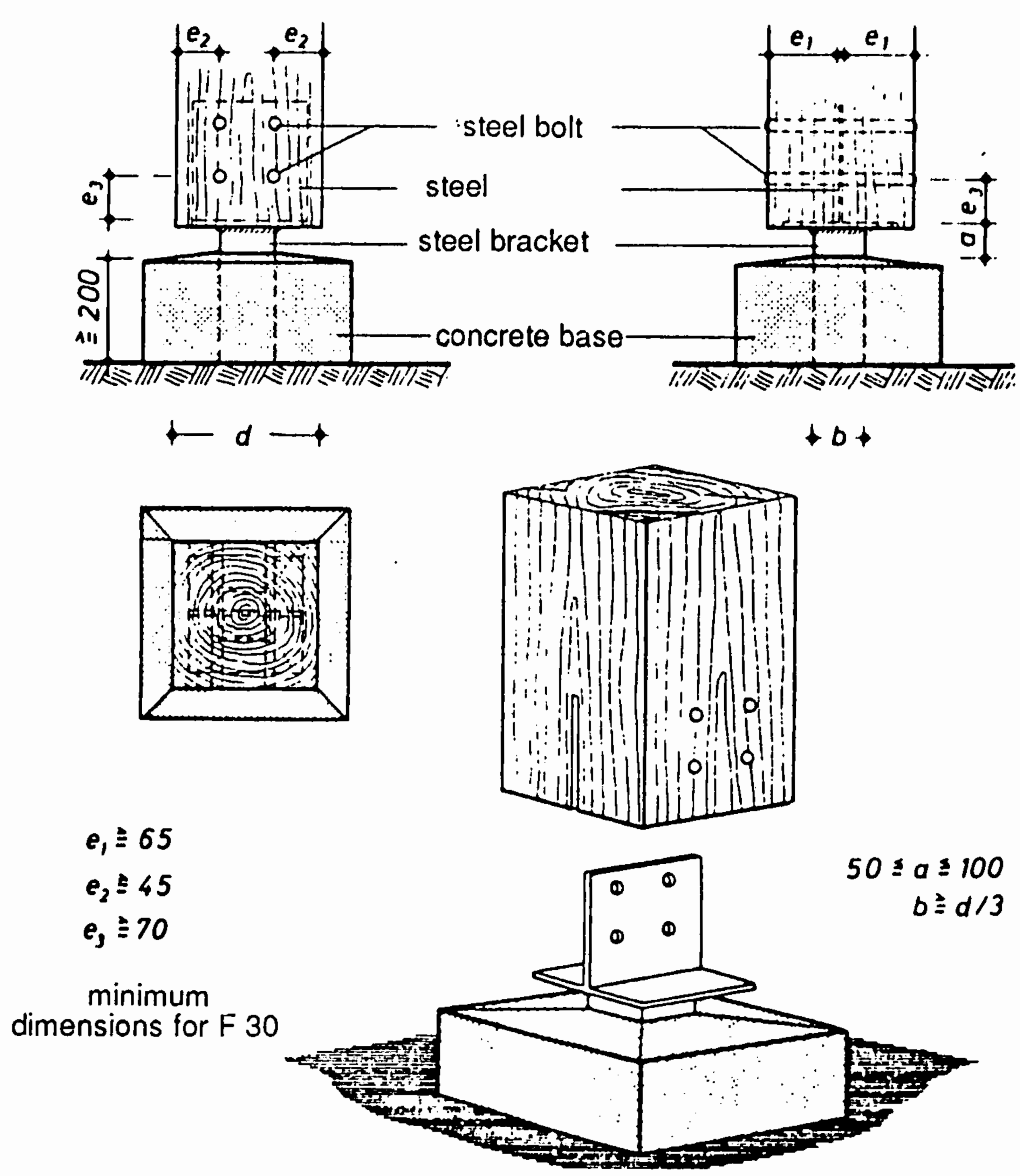


Figure 8.2: Column base for exterior use [8.1] in Fire Class F30.

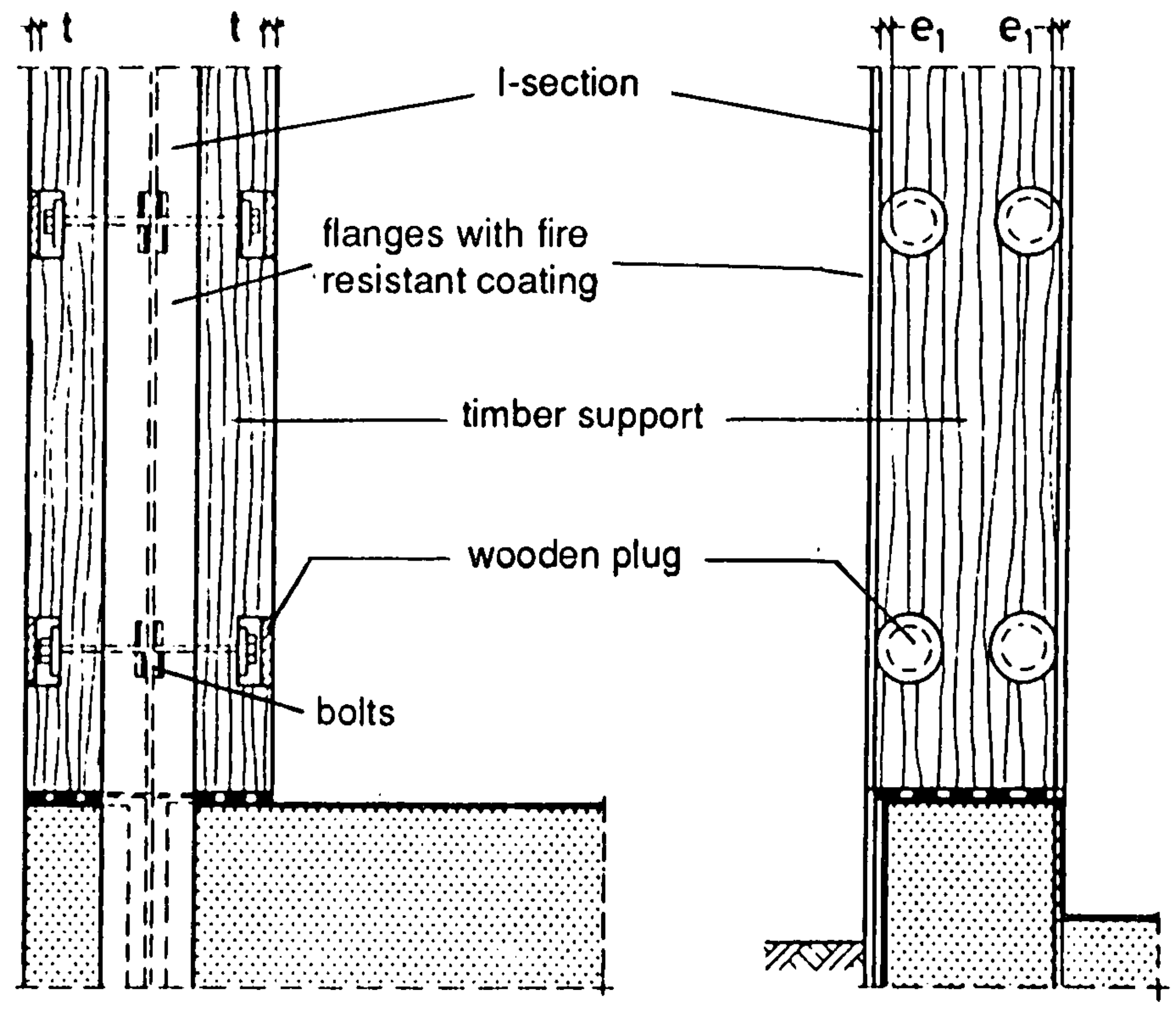


Figure 8.3: Fixed column in fire class F30. The flanges of the steel section had been treated with a fire-resistant coating [8.1].

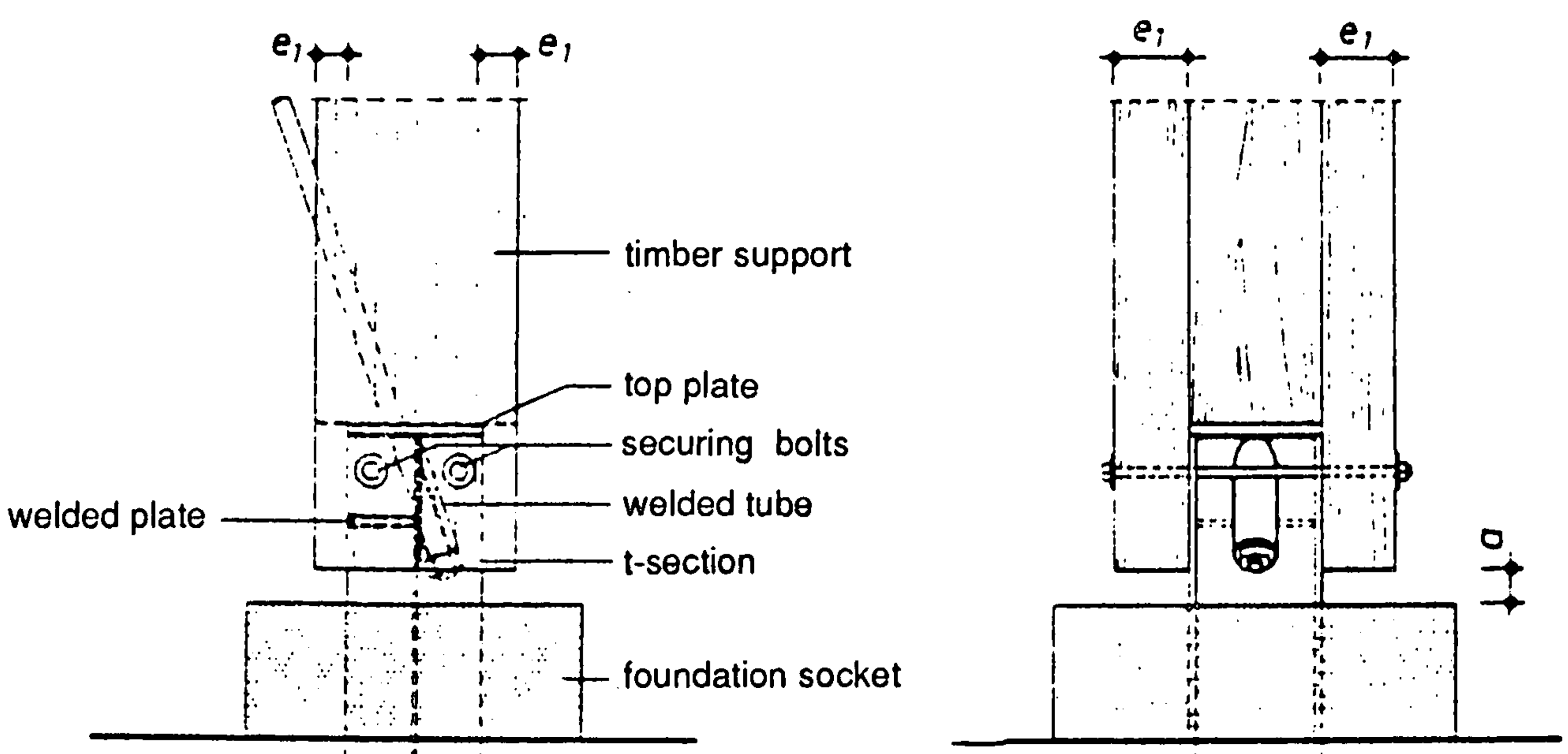


Figure 8.4: Column base joint [8.1] which satisfies the requirements of F30 if $50 \leq a \leq 100$ mm and $e_1 \geq 40$ mm. The bracing and its anchorage must be suitably protected.

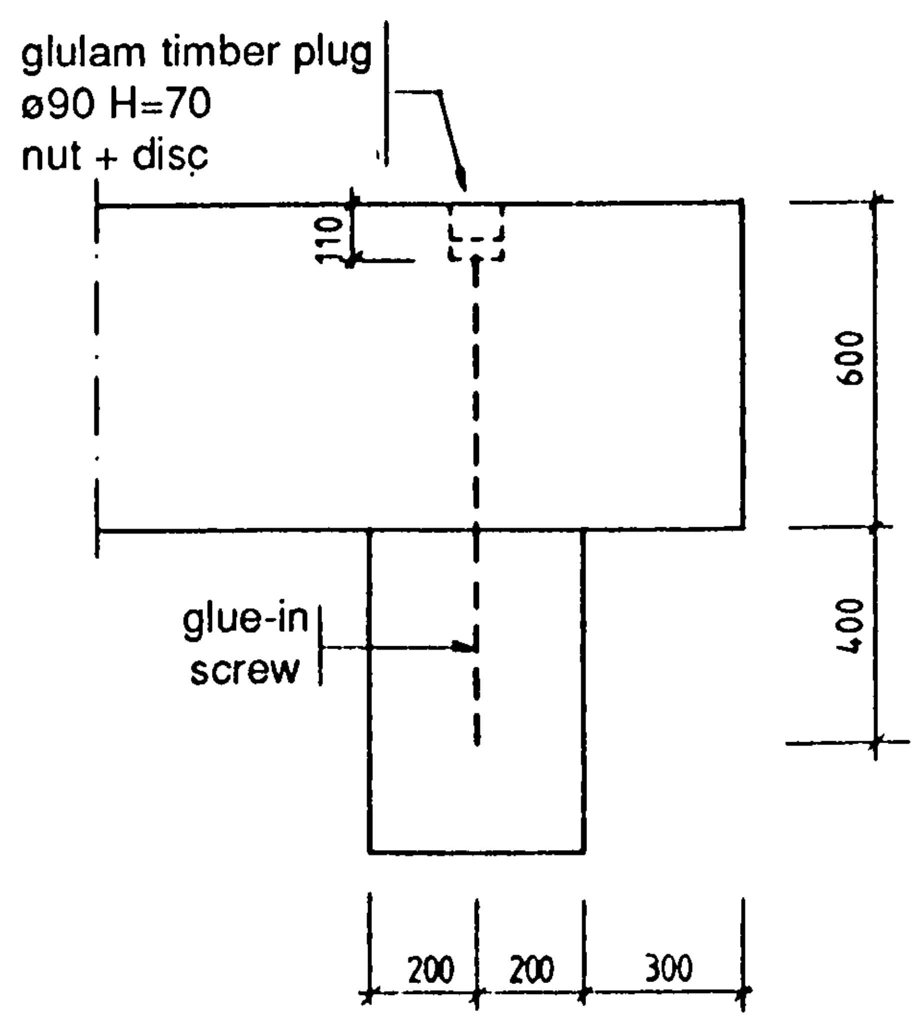


Figure 8.5: Fire tested beam-to-column joint with glue-in screw [8.4].

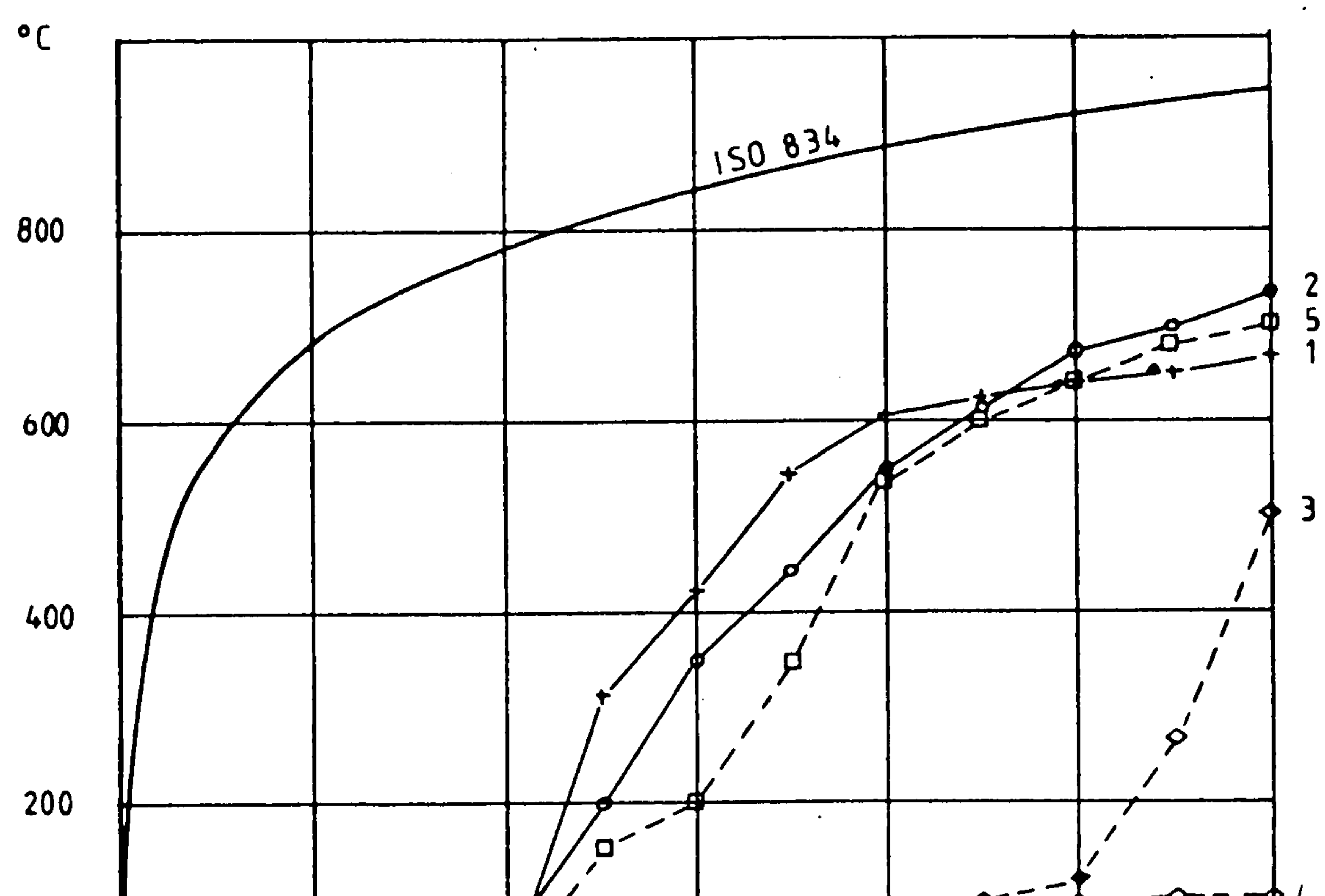


Figure 8.6: Temperature of the screw (curve 1) and at the bearing surface (curve 2) in the fire test of beam-to-column joint as shown in Figure 8.5. Temperature at the interface between the timber gusset and the side of the beam (curves 3 and 4) and the bearing surface (curve 5) in the fire test of beam-to-column joints as shown in Figure 8.7 [8.3].

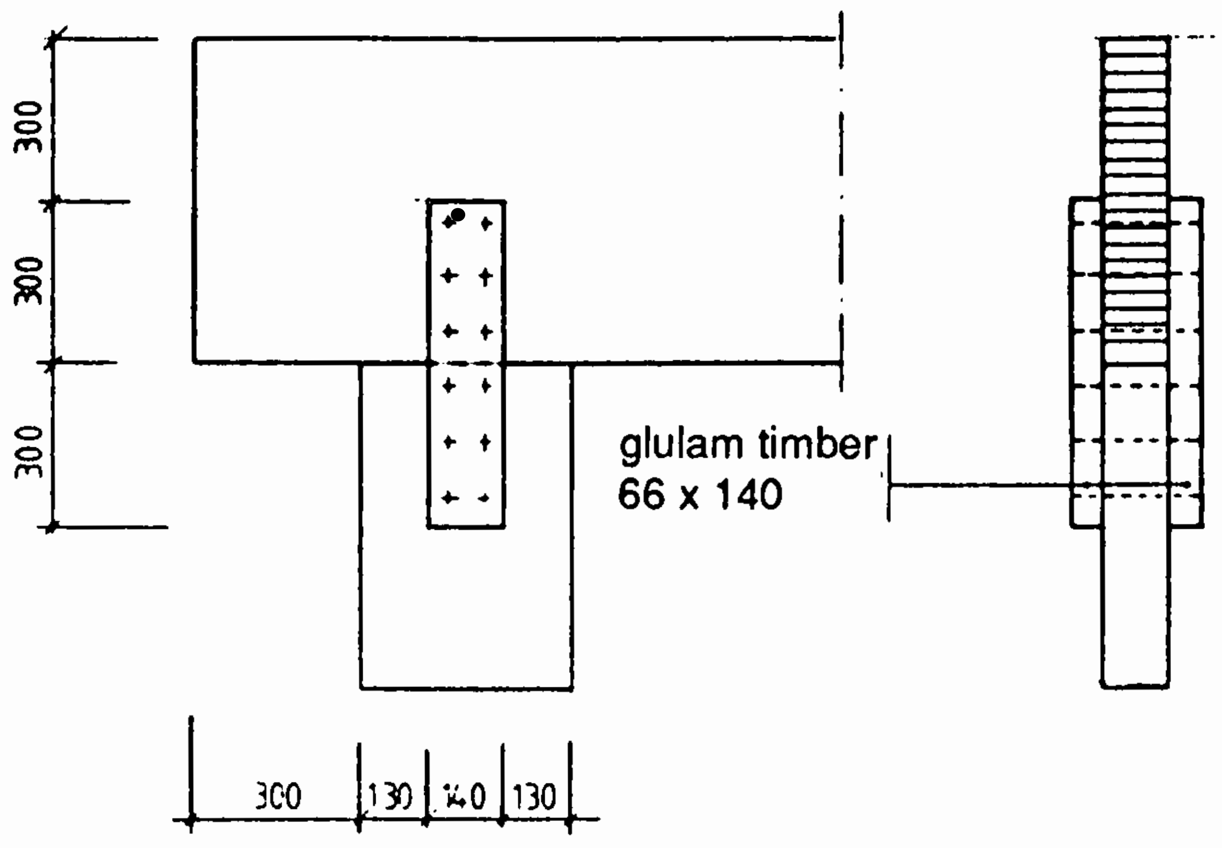
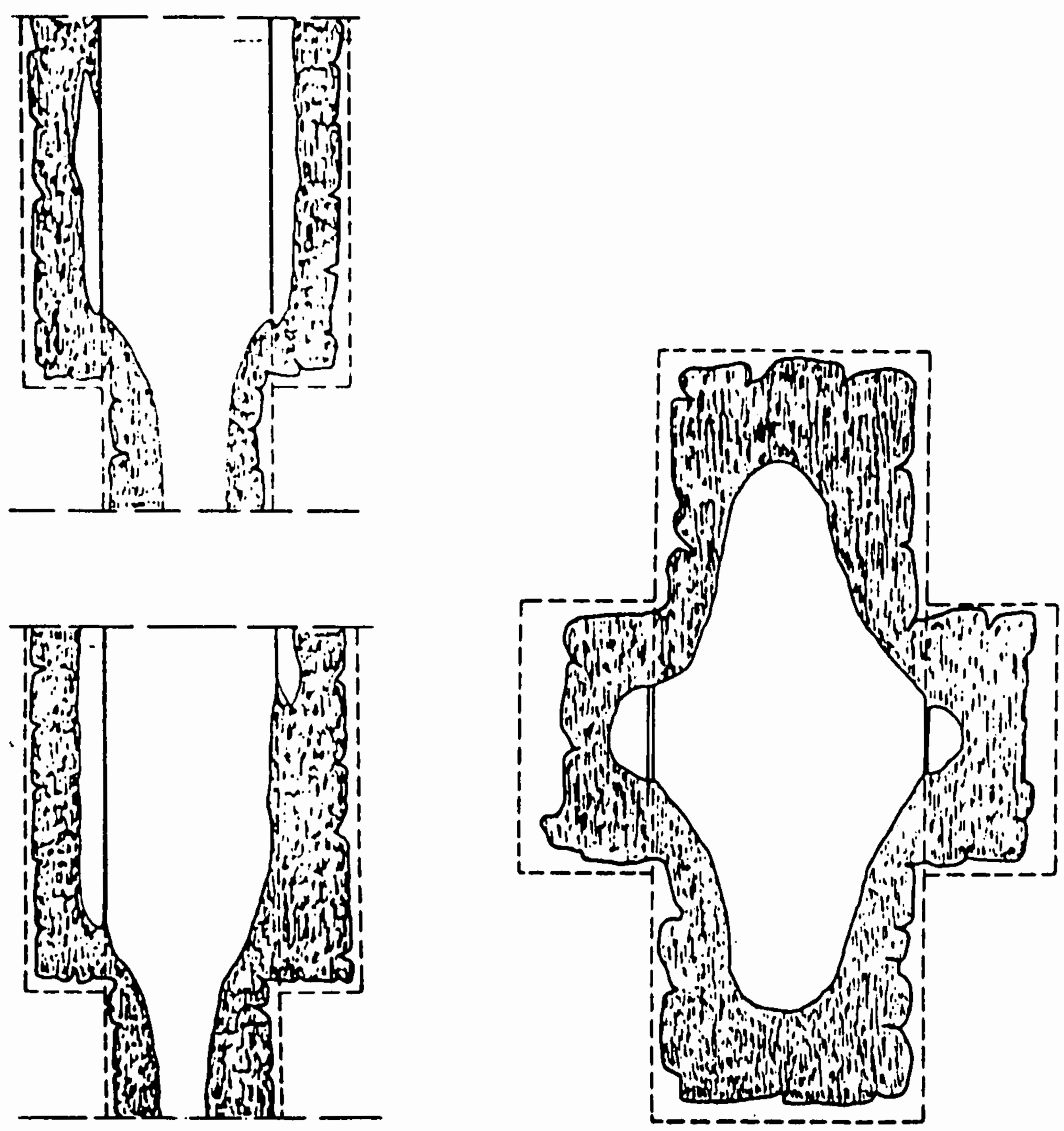


Figure 8.7: Fire tested beam-to-column joint with glulam timber gussets.



Vertical section

Horizontal section

Figure 8.8: Vertical and horizontal sections through the beam-to-column joint as shown in Figure 8.7 after 60 minutes of standard fire [8.4].

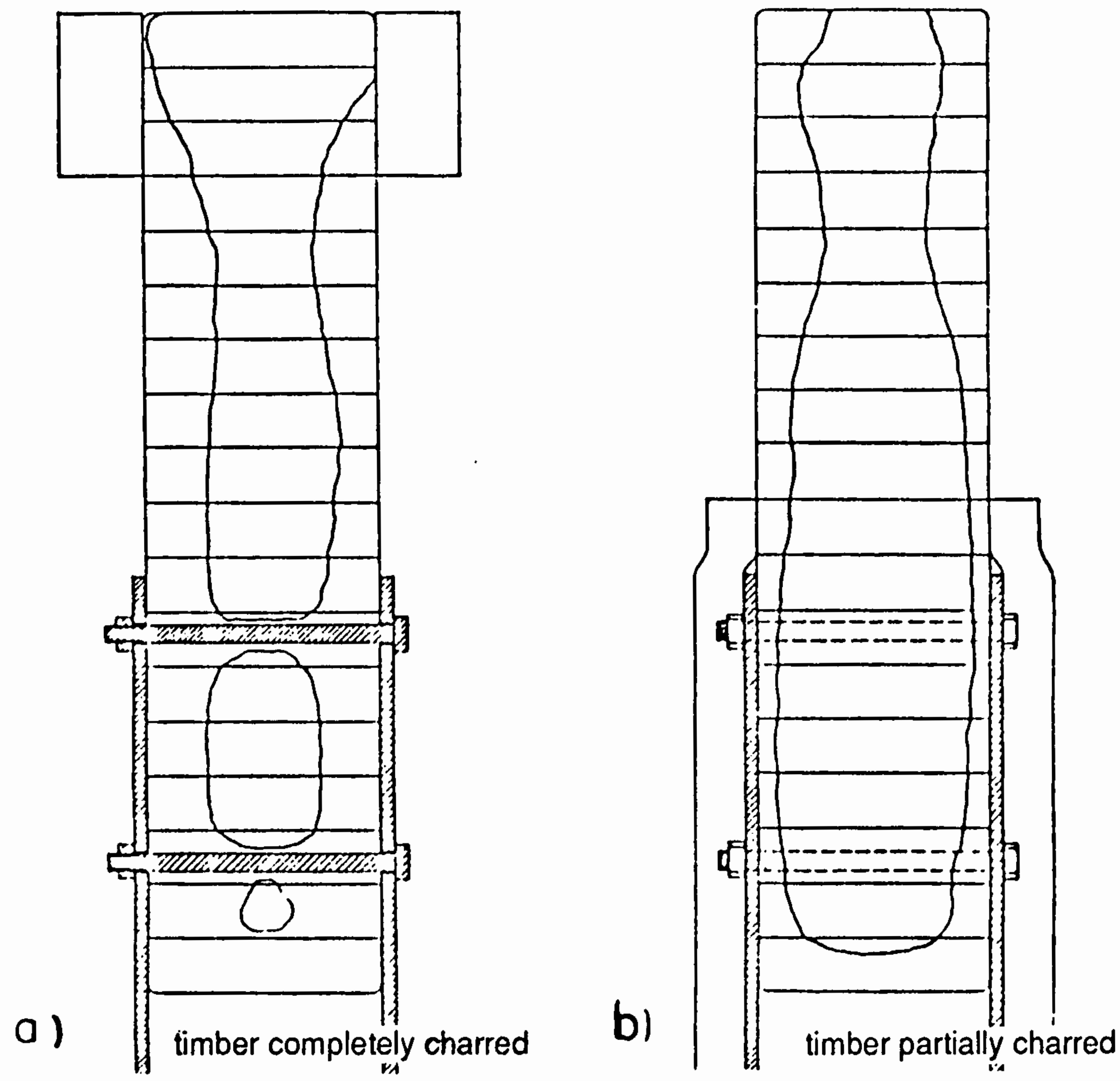


Figure 8.9: Vertical sections through the joint in Figure 7.5 after 60 minutes of standard fire [8.6], a) without protection, b) with fire protection.

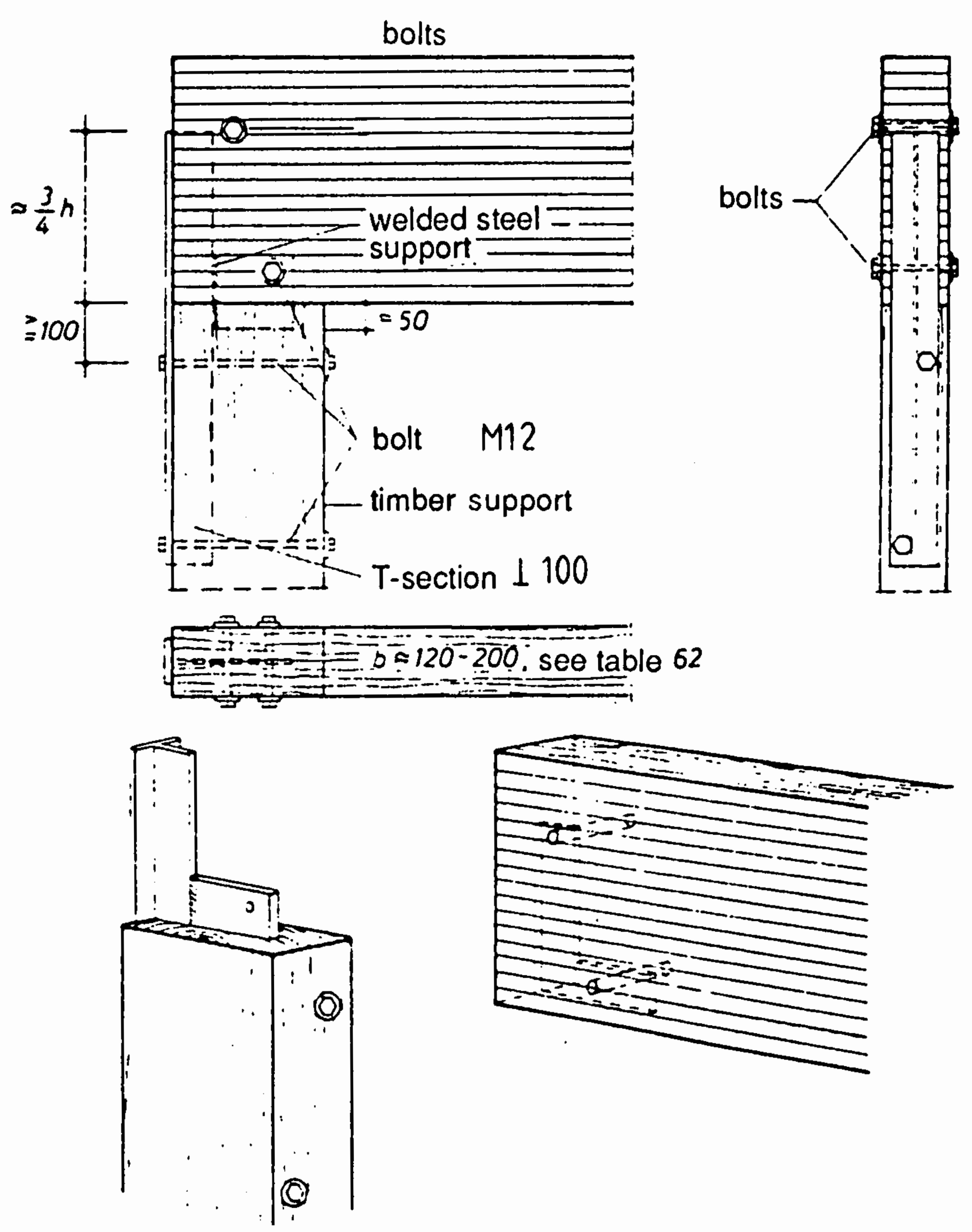


Figure 8.10: Beam-to-column joint with inserted steel T-section and classified according to [8.7] as F30 or F60 depending on the width of the beam. The column can be made of glulam timber or concrete.

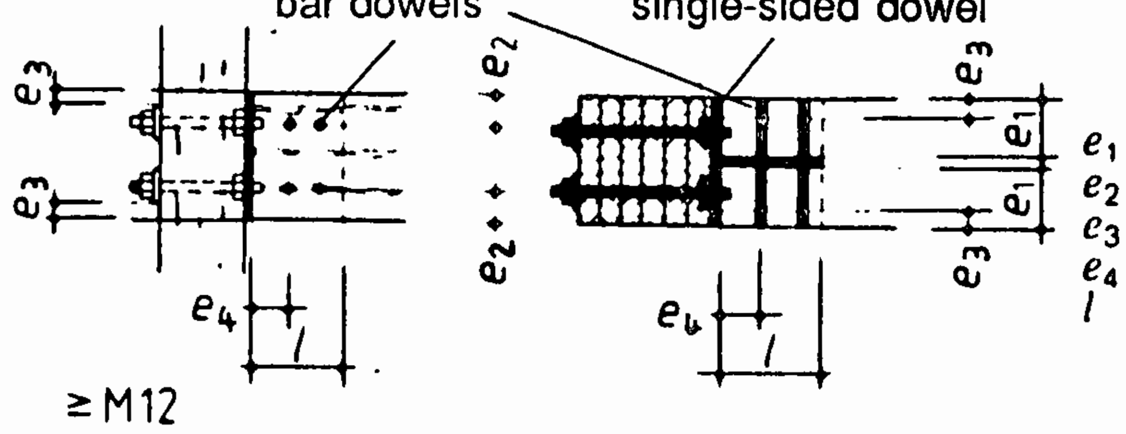
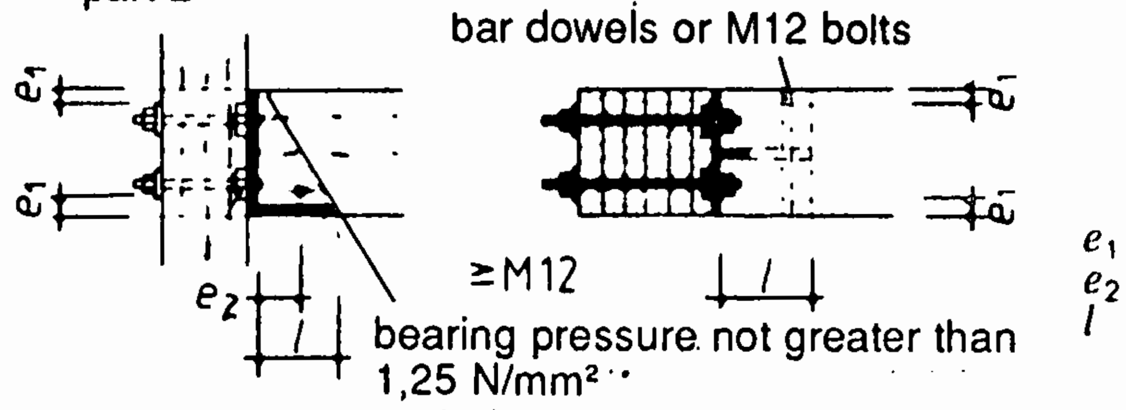
line	Structural features of the timber building components to be joined Dimensioins in mm	Minimum dimesions, in mm, for fire resistance class F30 - B, when using laminated timber	F30 - B, solid timber	F60 - B, laminated timber	F60 - B, solid timber
6	One-piece columns; fixing of beams using a piece of steel section with a thickness of not less than 10 mm.				
6.1	⊥- steel sections with dowels in accordance with DIN 1052 part 2  e_1 e_2 e_3 e_4 l	70 55 45 70 150	– – – – –	– – – – –	– – – – –
6.2	L- Steel angles with dowels in accordance with DIN 1052 part 2  e_1 e_2 l single-sided dowel in accordance with DIN 1052 part 2	45 70 100	– – –	– – –	– – –

Figure 8.11: Beam-to-column joint with steel support, bolted to glulam timber column and classified according to [8.7] as F30.

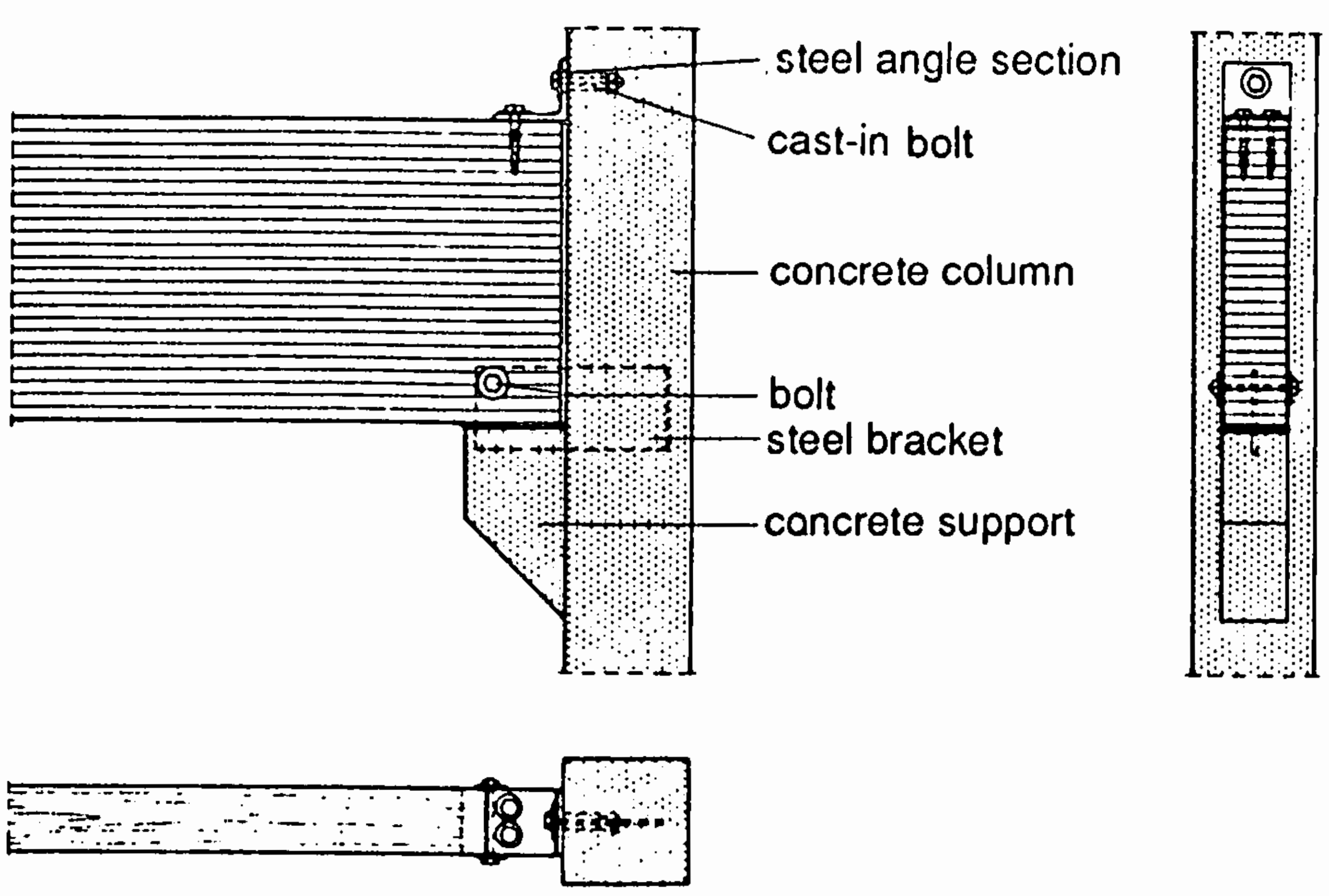


Figure 8.12: Beam-to-column joint with beam support made of concrete and side restraint of angle section is classified according to [8.7] as F30.

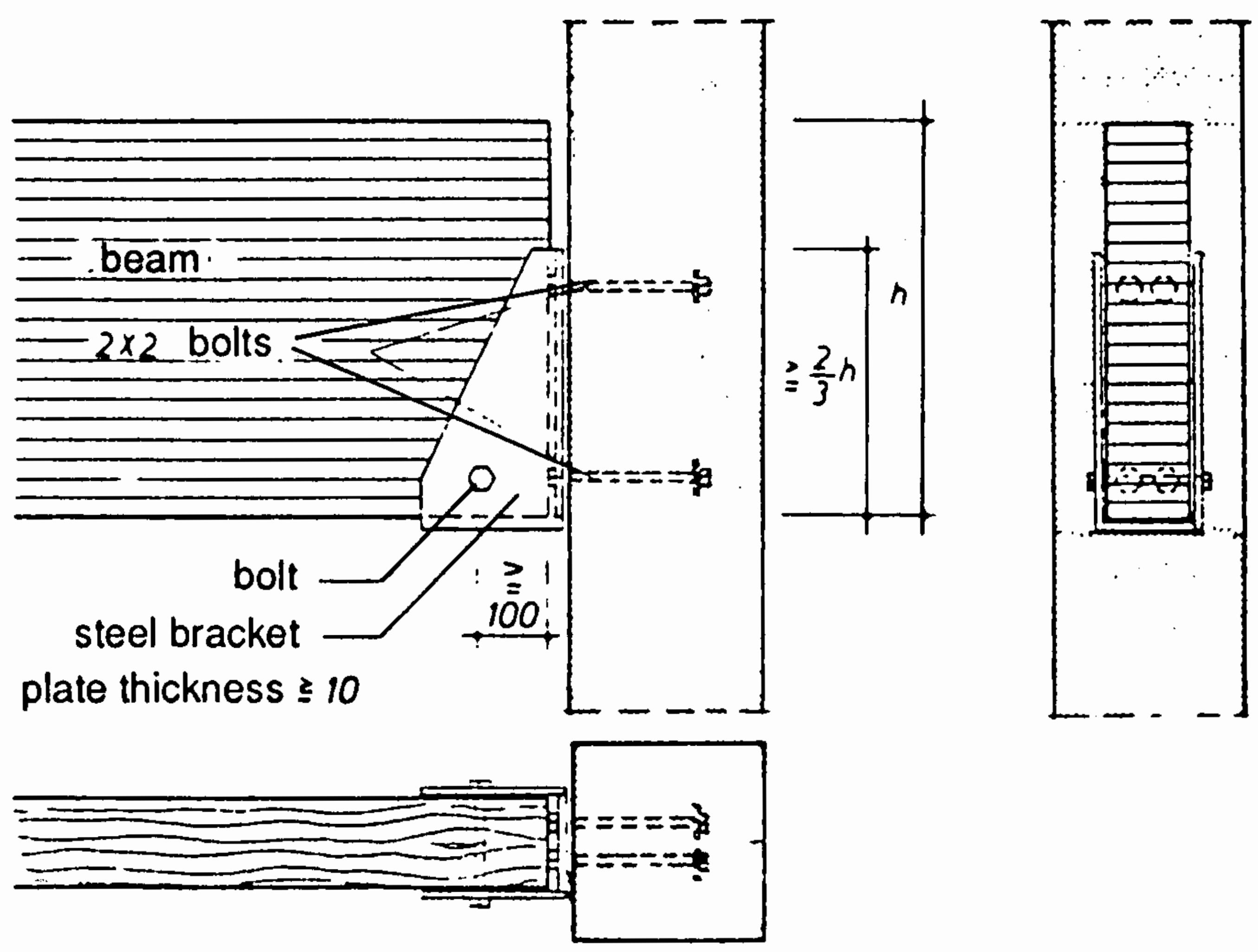


Figure 8.13: Beam-to-column joint with steel support bracket of minimum thickness of 10 mm, bolted to concrete or glulam timber columns which satisfies the require ments [8.1] for class F30.

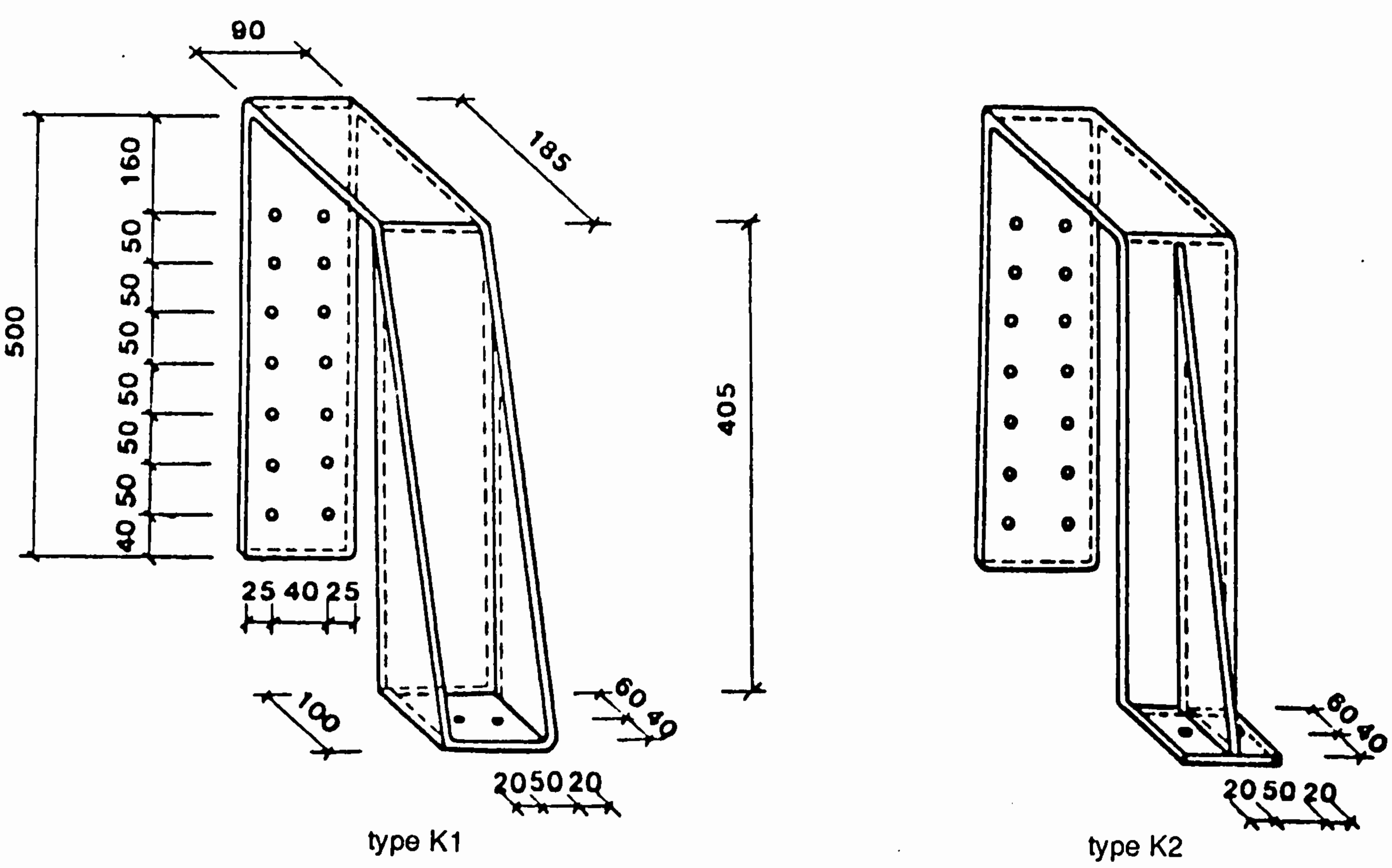
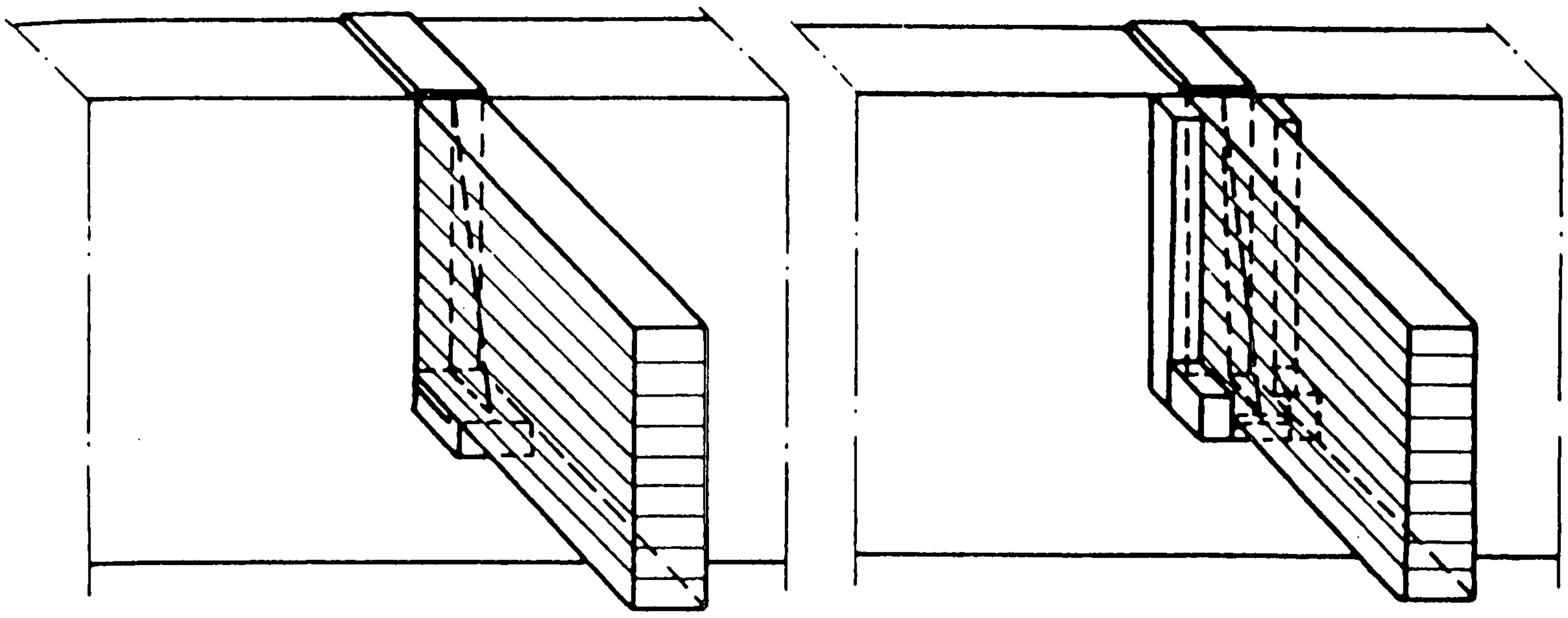


Figure 8.14: Fire tested fittings for joints between secondary glulam timber beam and primary glulam timber beam [8.4].



joint K3 joint K4

Figure 8.15: Fire tested alternative protections for fitting (type K2) of joints with secondary beams [8.4].

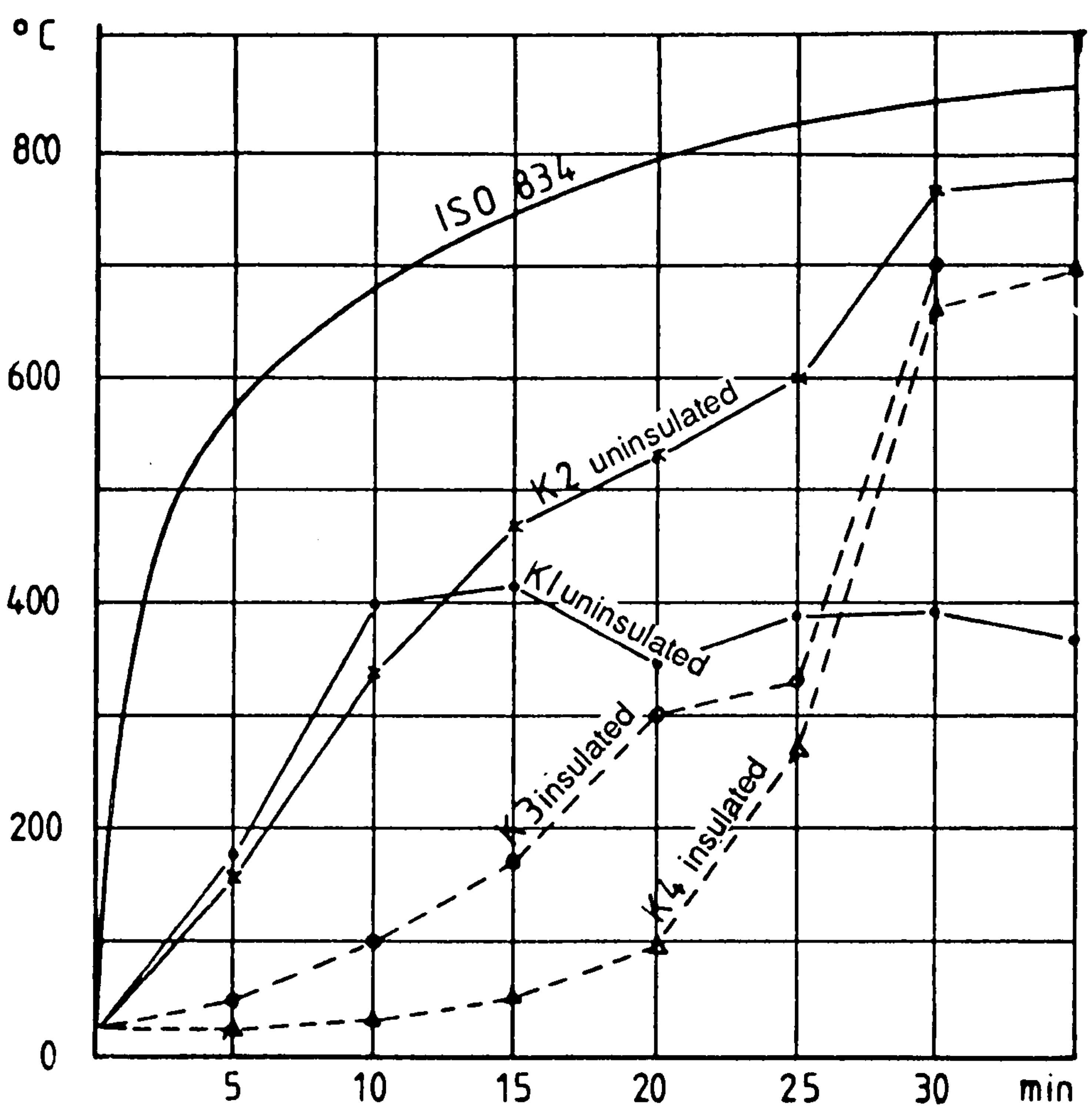


Figure 8.16: Temperatures at contact surface between secondary beam and support bracket in Figures 8.14 and 8.15 [8.3].

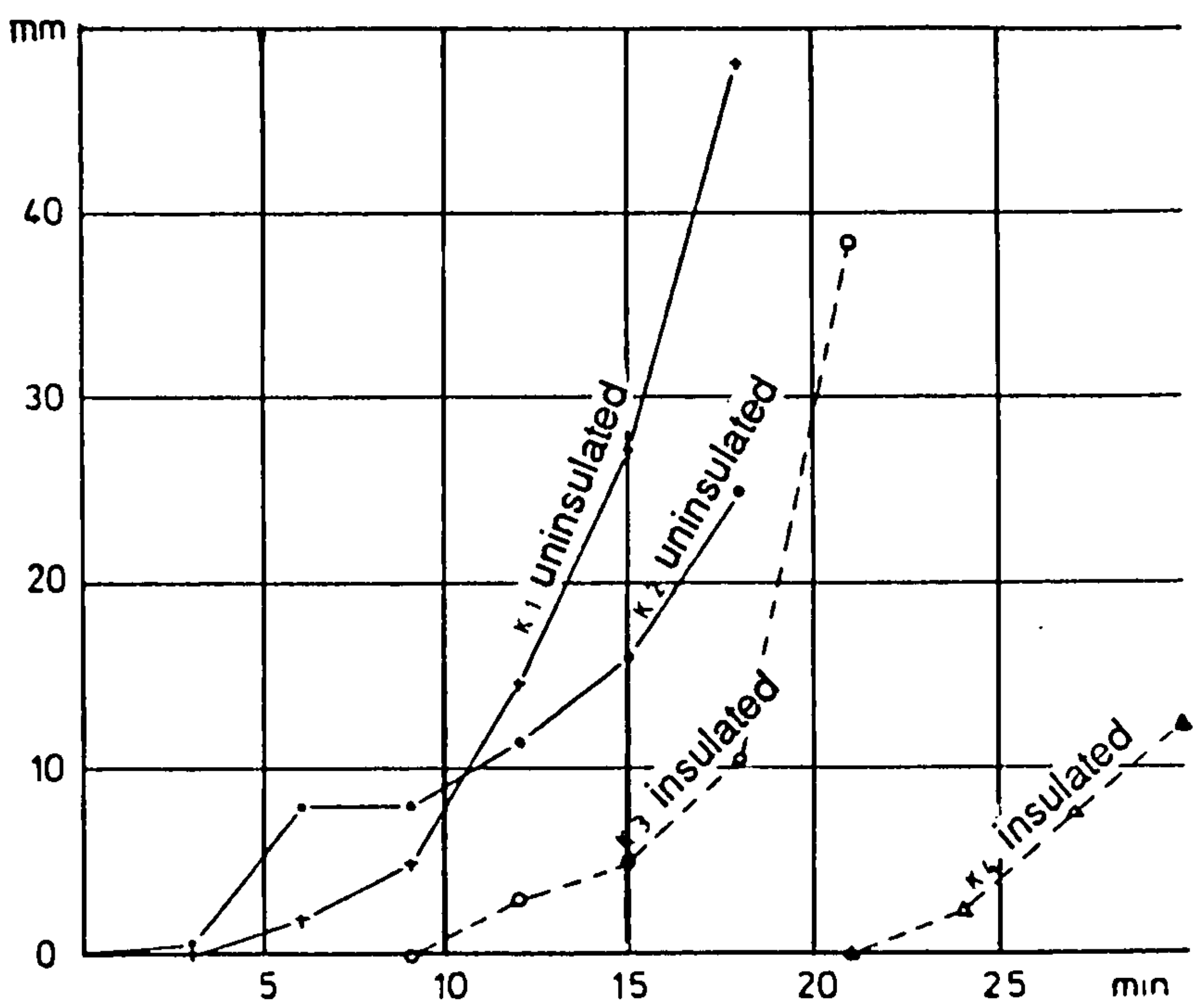


Figure 8.17: Downward displacement of secondary beam relative to primary beam in fire tests of support brackets in Figures 8.14 and 8.15 [8.3].

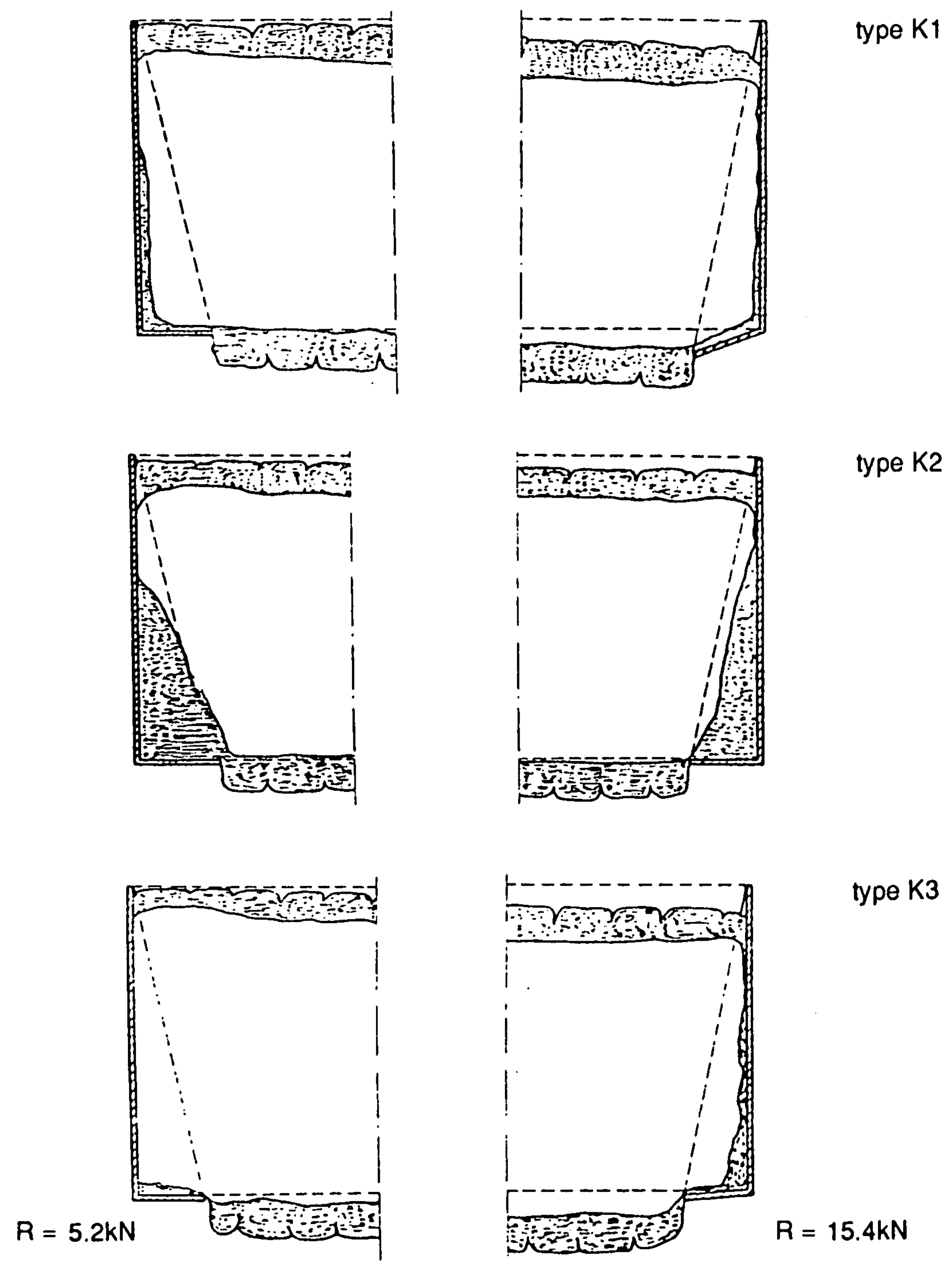
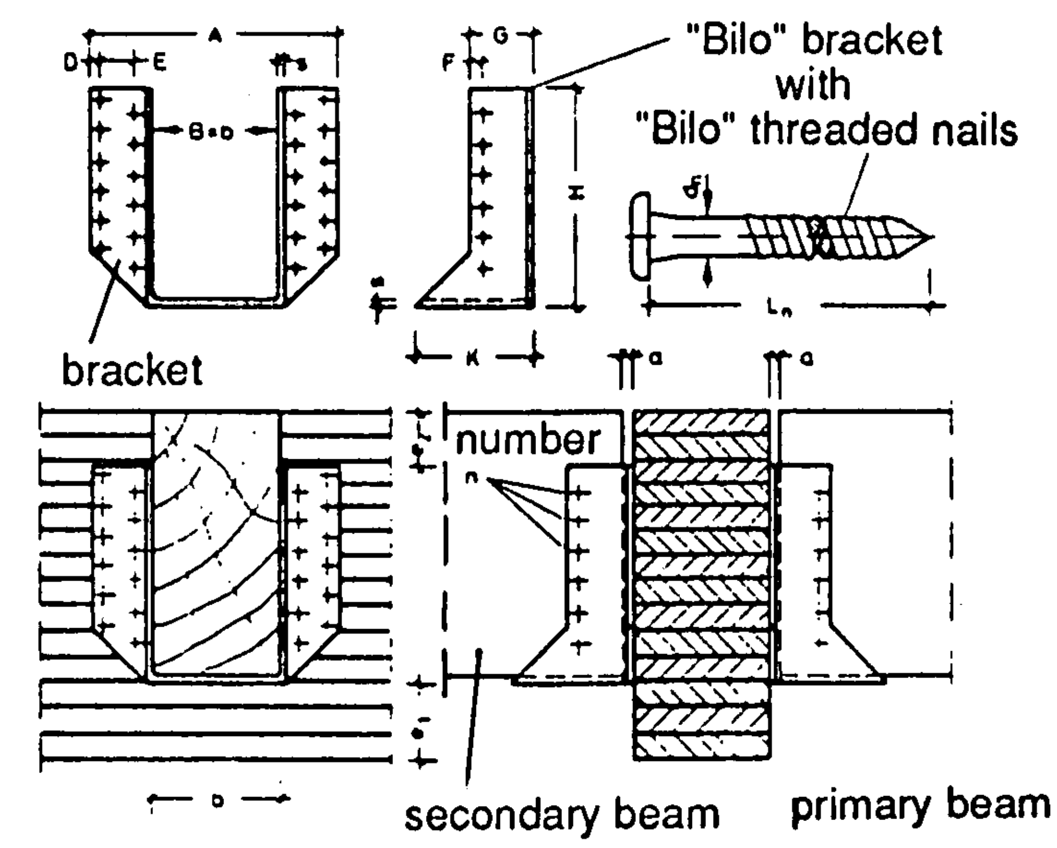


Figure 8.18: Vertical sections (along the beam) through the details in Figures 8.14 and 8.15 after 30 minutes of fire test [8.4].



1	2	3	4	5	6
Fire protection technology, important information			Other aspects refer to reference	Edge conditions for F30	
line	Bracket type	Threaded nail	Connection		Variation	Variation
1	N			Approved clause number Z 9.1-80 des IfBt	200 mm	170 mm
2	B-h				120 mm	100 mm
3	G				44 mm	40 mm
4	K				85 mm	75 mm
5	S				2 mm	2 mm
6		dn		Approved clause no.Z 9.1-61 des IfBt	4 mm	4 mm
7		Ln			75 mm	75 mm
8		N			0.75 permissible	0.33 permissible
9			b	[153] [154] [155]	120 mm	100 mm
10			n		2 . 7	2 . 6
11			n		2 13	2 12
12			d		3 mm	3 mm
13			e1		100 mm	50 mm
14			e2		30 mm	20 mm

permissible force = 0.75 kN per nail

Figure 8.19: Support brackets type BMF which were fire tested in West Germany [8.2] and classified as F30 [8.7].

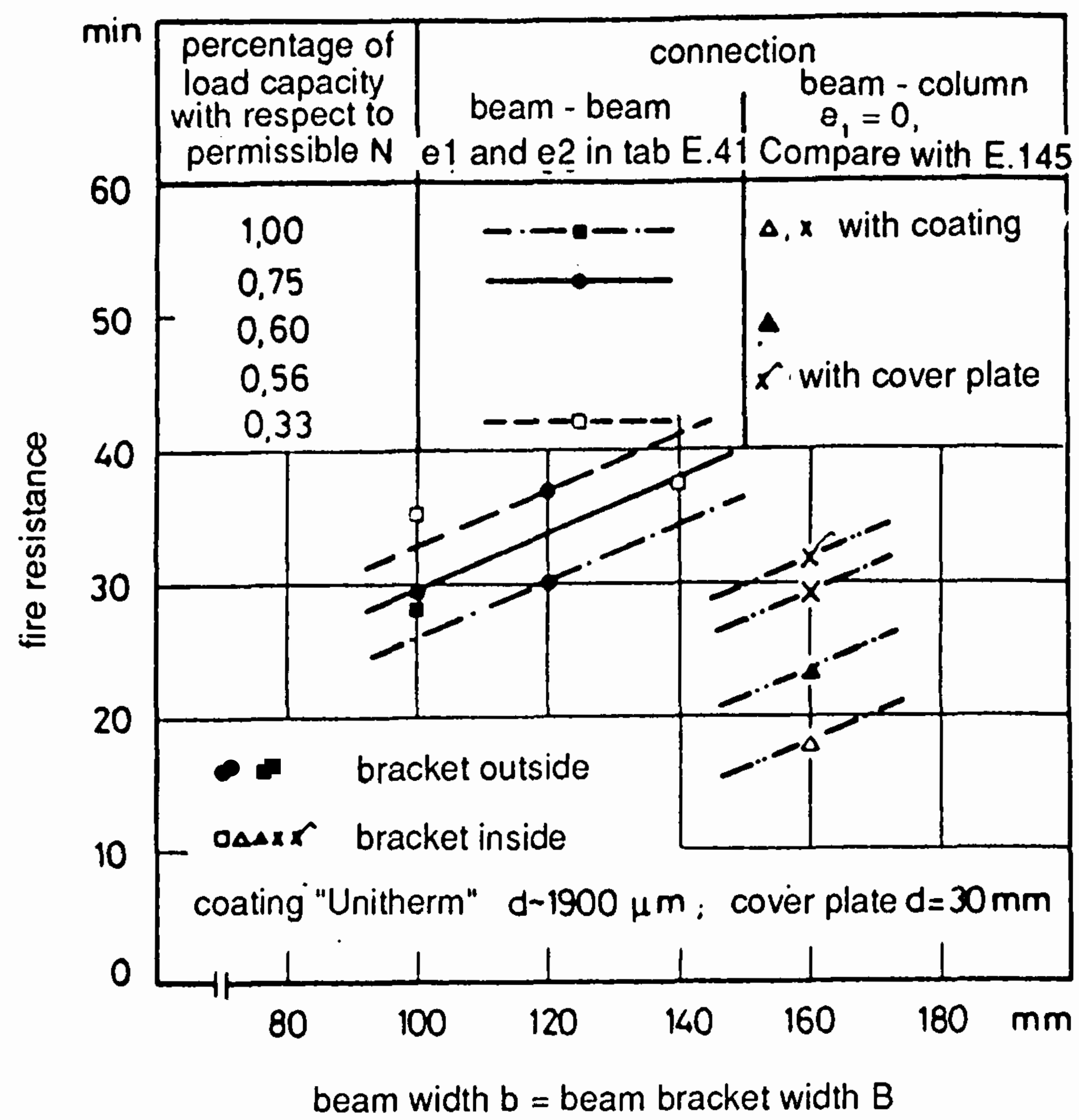


Figure 8.20: Summary of results from fire tests of beam support bracket type BMF [8.2].

Table 64: partially protected girder connection F30 - B

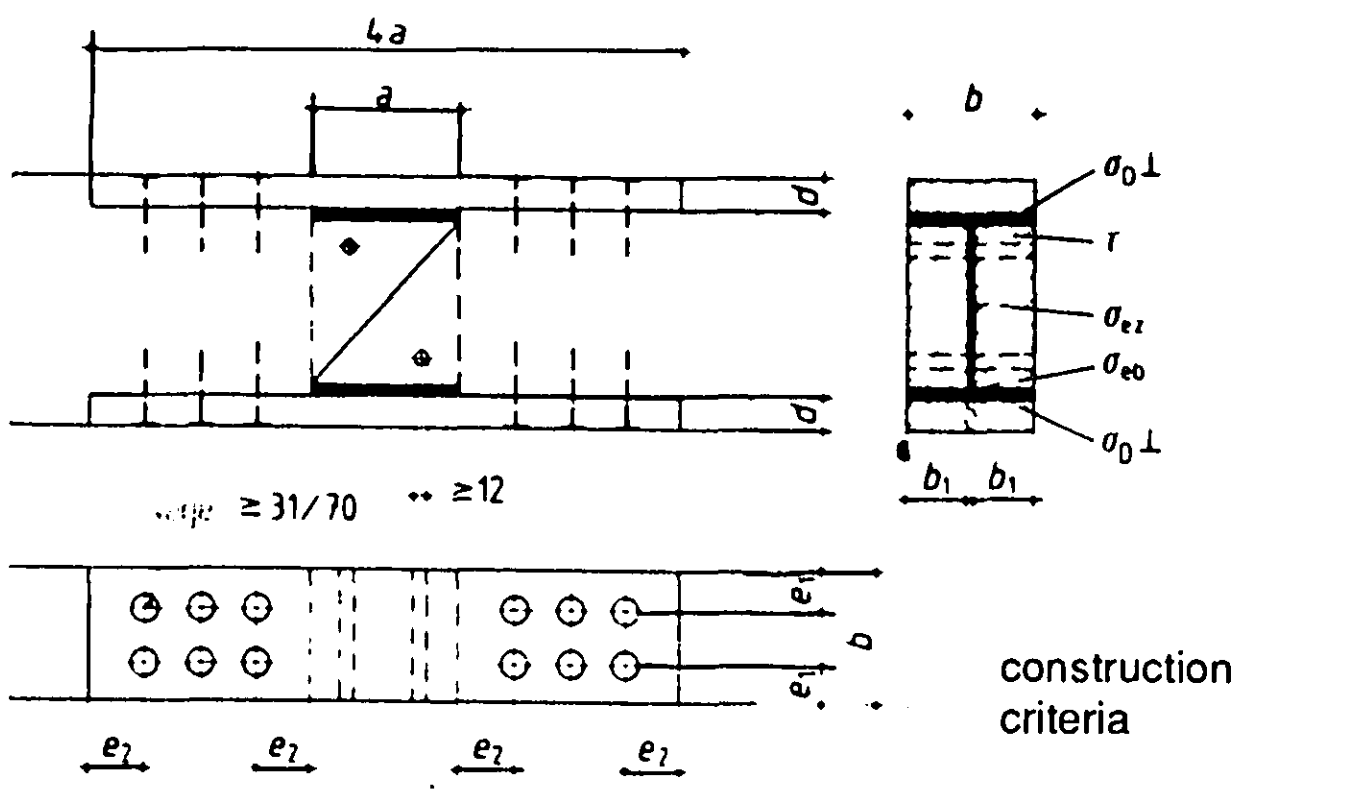
line	 construction criteria	Minimum requirement of use: laminated timber	solid timber
1.	**Minimum cross-sectional dimensions and minimum number of nails**		
1.1	Minimum beam width b, as long as it does not exceed the value given in Tables 61 and 62.	120	140
1.2	Minimum width b_1,	55	65
1.3	Minimum splice thickness d	30	30
1.4	Minimum edge distances e_1, and e_2	35	35
1.5	Minimum number of nails per splice	6	6
2.	***Permissible stresses in N/mm²**		
2.1	Maximum shear stress τ in timber	1.0 permissible - DIN 1052 part 1	
2.2	Maximum bearing stress (from steel, stress perpendicular to fibre)	1.6	
2.3	Maximum bending strength of flange	1.0 permissible - σ DIN 1050	
2.4	Maximum tensile strength of steel web and the welded joint	0.25 permissible - σ DIN 1050	

Figure 8.21: Girder connection, partially fire protected with timber materials classified according to DIN 4102 in F30 [8.7].

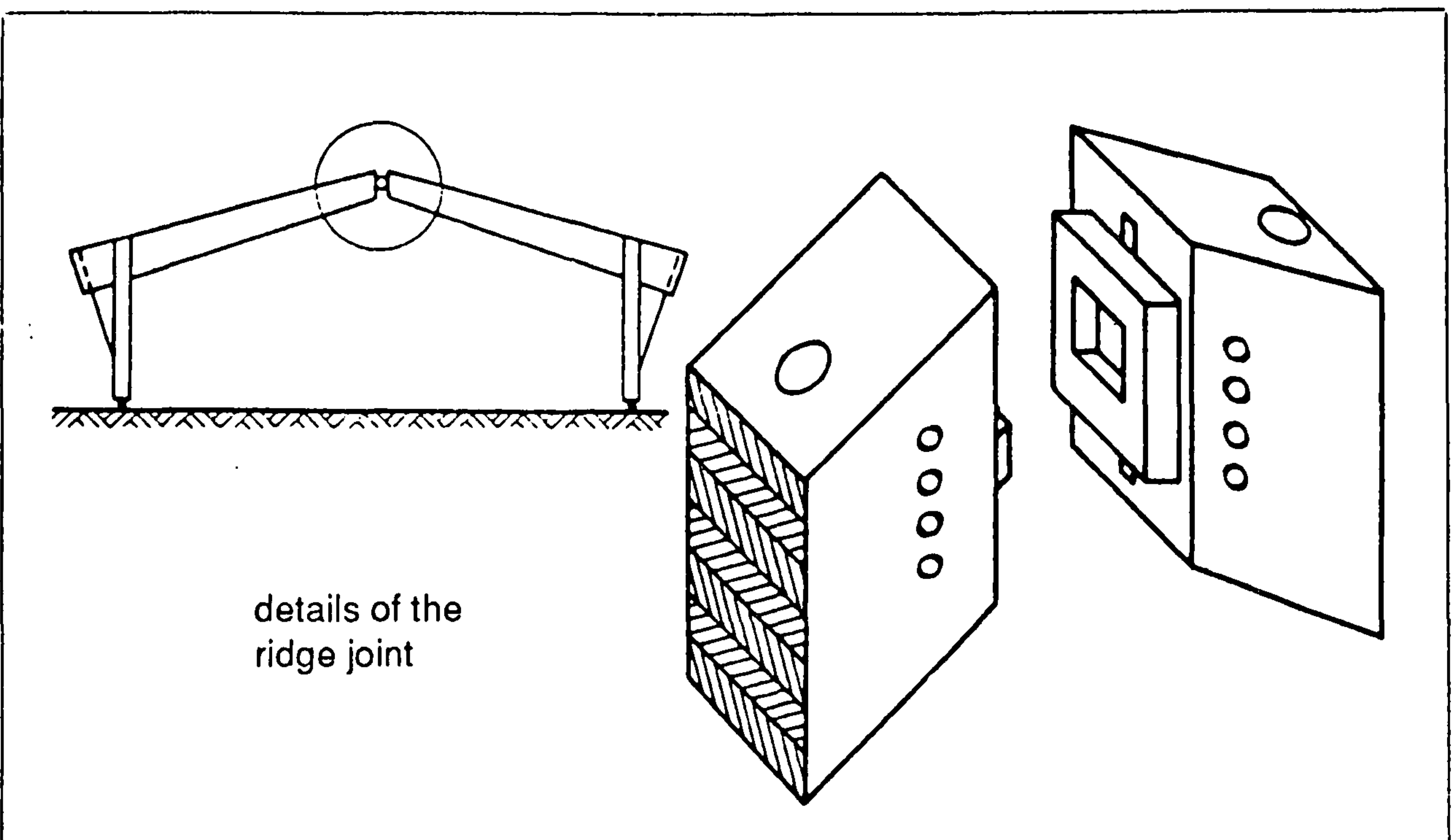
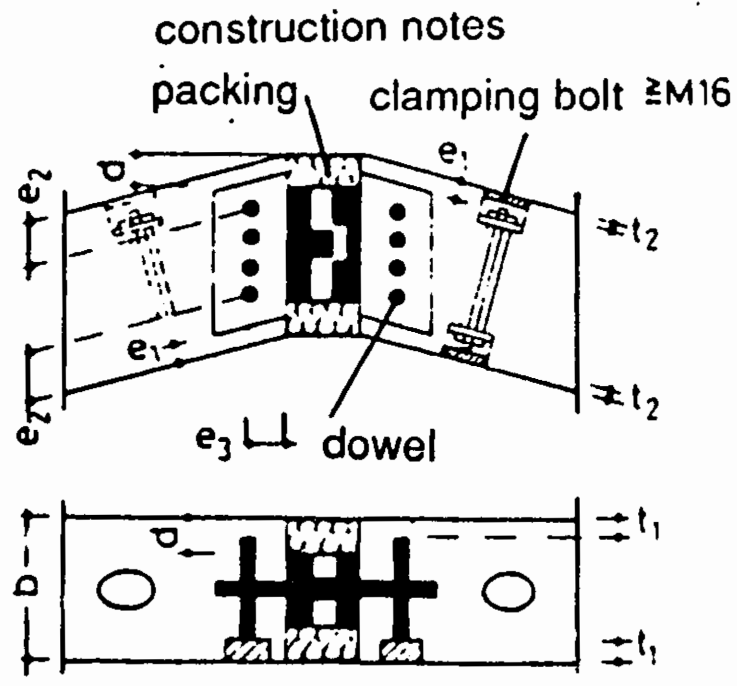


Figure 8.22: Fire tested hinged ridge joints which under certain conditions fulfill the require ments of F30 and F90 [8.1].

test no.	glulam timber beam cross-section b x h	ridge fittings: thickness mm	flange thickness mm	flange width mm	fire protection	bolt M 16: length mm	timber cover thickness mm	fire resistance min
1	120x400	10	10	100	-	120	-	35
2	120x400	10	10	100	-	120	-	31
3	120x400	11	11	100	Lita-flex KG 25	120	-	47
4	160x400	11	11	110	Lita-flex KG 25	160	-	58
5	200x400	10	20	180	-	120	40	54
6	200x400	10	20	180	Lita-flex KG 25	120	40	98

Table 8.4: Fire resistance of ridge joints in Figure 8.22 with different cross-sections and protective measures [8.2].

construction notes



			Minimum cross-section dimensions and maximum permissable stress for the following fire classes: F30 - B	F90 - B
1	Beam width	b(mm)	120	200
2	Timber cover over the steel section	e_1 (mm)	30	30
3	Timber cover over steel dowel			
3.1	according to over and under	e_2 (mm)	80	100
3.2	to beam front face	e_3 (mm)	80	100
3.3	sideways (lateral)	t_1 (mm)	not necessary	30
4	Timber cover over the clamping bolt		30	30
5	Packing of "Litaflex kg 25", glued with "Litaflex-Kleher 800" adhesive and compressed to about 1cm (on all sides)	d (mm)	not necessary	70
6	Permissible stress (N/mm2)			
6.1	τ		permissible τ	0.67 permissible τ
6.2	$\sigma\perp$ (initial jointing stress)		0.9 permissible σ	0.45 permissible σ

1) Obtained from "Firma Rex Asbestwerke, Schwabisch Hall"
2) Permissible τ and σ according to DIN 1052

Figure 8.23: Conditions under which hinged ridge joints in Figure 8.22 fulfill the require ments of F30 and F90 [8.1].

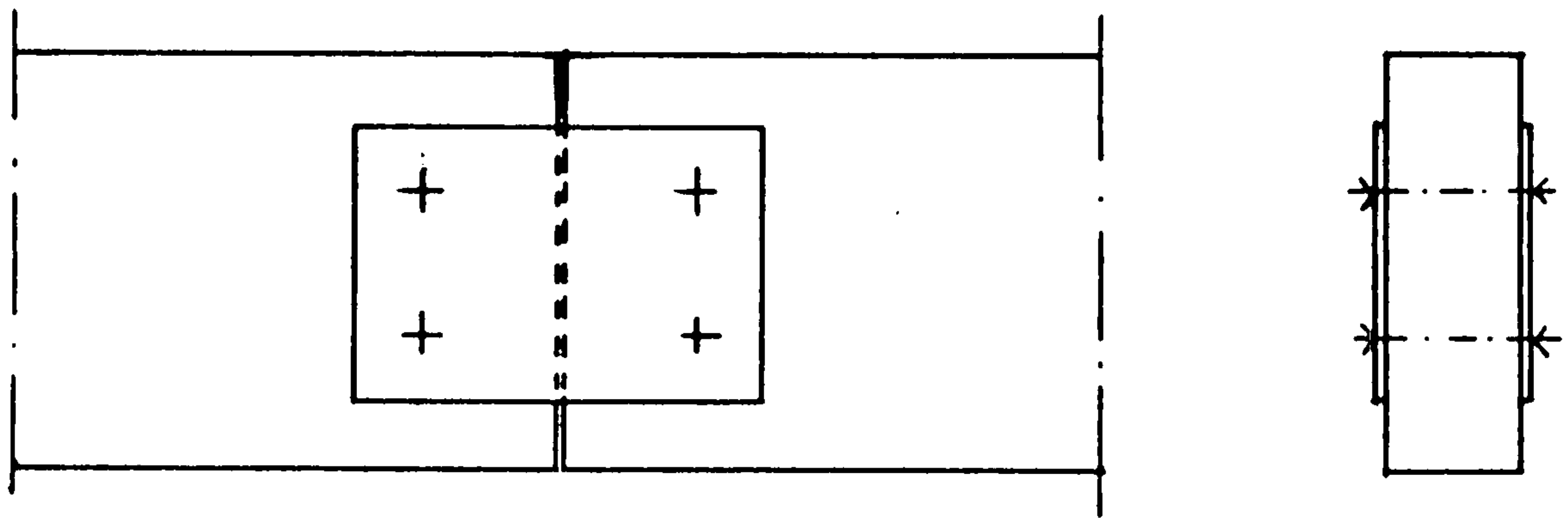


Figure 9.1: Fire resistances of a bolted timber joint and the principles for their calculation [9.4], [9.5].

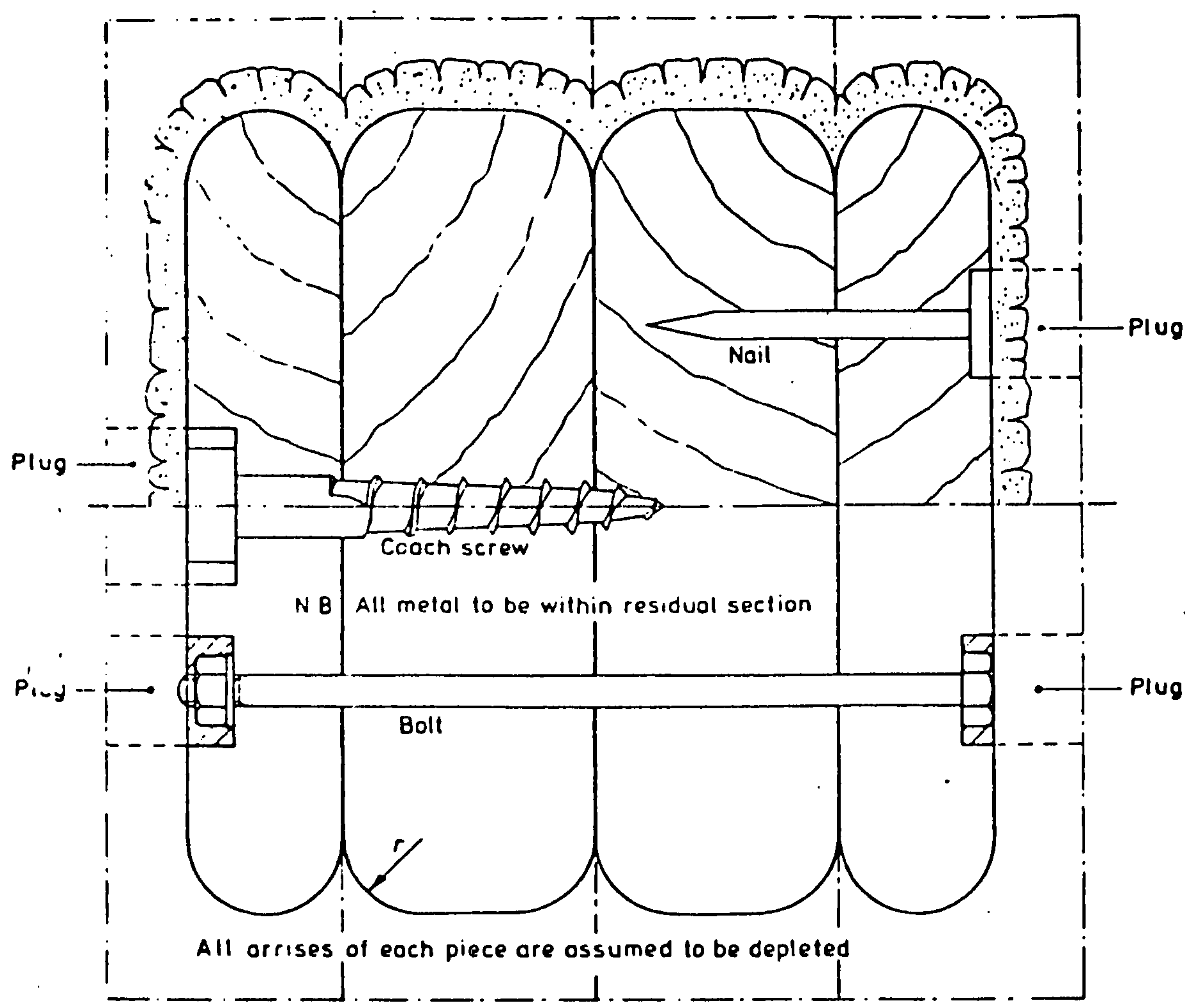


Figure 10.1: According to BS 5268, a metal connection should be buried into the timber so that it is completely within the residual section.

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