

Holistic behaviour of concrete buildings in fire.

Professor Colin Bailey
Manchester Centre for Civil and Construction Engineering
PO Box 88
Manchester
M60 1QD
UK

This paper was originally published in the Proceedings of the Institution of Civil Engineers, Structures and Buildings 152, August 2002, Issue 3, pp 199-212. It is republished here with kind permission of Thomas Telford Limited.

Abstract

This paper discusses various modes of structural behaviour of a concrete building when subjected to a fire, based on observations from a full-scale test. Although some data were lost during the test, the available results and observations presented provide a valuable insight into the holistic behaviour of concrete buildings, when subjected to fire. The tested building was constructed using elements formed from normal and high-strength concrete and was designed for 60 minutes fire resistance, using the UK design Code. High-strength concrete was used for the columns within the fire compartment and since it has previously been shown that this type of concrete is susceptible to spalling, polypropylene fibres were added to the concrete mix during construction to alleviate the problem. Both the UK and European codified design methods suggest that concrete spalling within the fire compartment should have been nominal and could effectively be ignored during the design. However the test showed that spalling of the floor slab was extensive and exposed the bottom steel reinforcement. Although concrete spalling considerably reduced the flexural strength of the slab, collapse did not occur. This could be attributed to the slab behaving in compressive membrane action, which is currently not considered in codified design methods. The test also showed significant lateral displacement of external columns due to thermal expansion of the heated slab. The main observations from the test show that designers will need to understand the behaviour of entire structures in fire, to ensure that premature collapse will not occur.

Key Words

Concrete structures, Thermal effects, Buildings, structure & design.

1. Introduction

Between July and September 2001 a unique opportunity presented itself to carry out a fire test on the full-scale, seven-storey, concrete building constructed at the Building Research Establishment (BRE) laboratories in Cardington, Bedford. The design and construction of the insitu-concrete building at Cardington was part of the European Concrete Building Project⁽¹⁾. This project is an industry led initiative, with the aim of improving efficiency of production and enhancing the performance of buildings using different forms of concrete construction.

The main aim of the fire test was to investigate the behaviour of a full-scale concrete framed building subjected to a realistic compartment fire and applied static design load. Prior to the fire test, it was envisaged that the results and observations obtained would contribute towards a wider proposed programme of research work, into the holistic behaviour of concrete buildings in fire. The ultimate aim of this research is to produce design guidance based on the realistic structural behaviour of the building as a whole, which will result in the construction of safer and possibly more economical buildings. The main objectives of the overall proposed research were identified as follows.

- To investigate how the building in its entirety resists or accommodates the large thermal expansions from the heated parts of the structure within a given fire compartment.
- To identify both beneficial and detrimental modes of whole building behaviour, that cannot be shown from standard small-scale fire tests.
- To investigate the overall effects of any concrete spalling and to determine its significance on the behaviour of the whole building.
- To compare test results and observations from large-scale fire tests with current methods of design.

This paper presents the results from the fire test on the Cardington building, which will address some of the above objectives. Unfortunately the test results are restrictive in that the fire destroyed the instrumentation cables during the test, leading to the loss of some data. However, the observations and results obtained are of interest and highlight both modes of beneficial and detrimental behaviour of the building as a whole, which are ignored in current design methods.

Before the observations and results from the test are discussed it is worth briefly reviewing current design methods, design assumptions and typical perceptions of the behaviour of concrete structures in fire. This review will allow the test results to be compared against current design assumptions.

2. Current design methods and perception

Structural collapse of concrete structures due to fire is rare, but not unknown. For example the following structures suffered total or partial collapse as a result of fire; the Katrantzos Sport building⁽²⁾ in Athens (1980), Linde Factory⁽³⁾ in West Germany (1971), Minin Works⁽⁴⁾ in Surrey, UK (1969), and a library building in Linköping⁽⁵⁾, Sweden (1996). Of course there are also examples^{(4),(6)} where concrete structures have retained their stability for the complete burn out of the fire. In the published cases of structural collapse, the cause has generally been attributed to poor continuity of reinforcement or poor workmanship during construction. Although this is a plausible explanation, recent thinking has highlighted detrimental behavioural aspects of concrete buildings, which are considered to be inadequately addressed in current design codes, and can lead to premature structural collapse. For example, it is thought that a common cause of collapse is due to the structure as a whole being unable to accommodate or resist the large horizontal displacements induced by the thermal expansion of heated floor slabs. Another example, not covered adequately in design codes, is the effect of spalling of concrete in fire^{(7),(8)}. The removal of cover to reinforcing bars by spalling could lead to premature failure of members or entire structures.

The current fire design^{(9),(10),(11)} of concrete structures is primarily based on member design with tabular rules and analytical methods derived from results of isolated beams, columns and slabs tested in small furnaces. The underlying assumption is that the fire resistance of the complete structure will be at least equal to the fire resistance of its individual members. However, the behaviour of the entire structure when considered as a collection of connected members is significantly different from the behaviour of individual isolated members, with both beneficial and detrimental effects occurring. By following the current design methods, the designer may be unaware that the approach assumes that the beneficial structural behaviour of the building, acting in its entirety, will outweigh any possible detrimental effects. If correct, this assumption will ensure that the fire resistance of the whole structure is at least equal to the fire resistance of the individual members used to construct the building. In some cases, by ignoring the true behaviour of the building, sufficient detrimental effects, which are not highlighted from member design and testing, may lead to premature

collapse of the structure. An obvious difference between whole building and member behaviour is that the structure will utilise actual load path mechanisms that cannot be identified from member testing. For example, restraint to thermal expansion of a floor slab, caused by a cooler part of the structure surrounding a heated compartment, can induce high compressive stresses into the heated slab, which can be beneficial by inducing compressive membrane action to support the applied load. These high compressive forces can also, however, increase the slab's susceptibility to spalling. Another form of whole building behaviour, that can lead to premature collapse, results from the inability of vertical members to resist or accommodate the large lateral movements caused by the thermal expansion of the heated floor slab. This form of behaviour is not covered in current design codes and has been recently highlighted by Van Acker⁽⁵⁾, as the reason for some structural collapses of concrete structures in fire.

Spalling of concrete in fire involves the breaking off of layers or pieces of concrete from the surface of the structure, as it is heated. Although a large amount of research^{(7),(8),(12),(13)} has been conducted into spalling, the behaviour is difficult, if not impossible, to predict and this is probably the reason why design codes provide no definitive guidance. The behaviour of spalling is typically classified into three types; aggregate spalling, explosive spalling and corner spalling. Aggregate spalling occurs in the early stages of the fire and involves small pieces flying off the surface. As the name suggests aggregate spalling is mainly caused by the break up of the aggregate at elevated temperatures, but typically results in only superficial damage. Flint and Thames gravel are particularly susceptible to this type of spalling^{(7),(10),(14)}. Explosive spalling, which is extremely violent, also occurs early in the fire, and is the most serious in terms of causing premature collapse of the structure. The consequence of explosive spalling has resulted in it being the most researched, with the main causes being attributed to heating rate, moisture content, permeability of the concrete and mechanical stress levels. High strength concrete has been shown to have a high susceptibility to explosive spalling and some measures⁽¹⁵⁾ to alleviate the problem need to be considered by the designer. Corner spalling is non-violent and occurs later in the fire and results in large sections of concrete at the corners of columns or beams breaking off. In real fires corner spalling will not significantly affect the exposed steel reinforcement, since the fire temperatures are typically low towards the end of the fire. However, in fire resistance tests on members where the temperatures continue to rise for the duration of the test, corner spalling can be significant, resulting in failure of the member.

The general perception that concrete structures have a good inherent fire resistance has typically resulted in most UK designers never venturing beyond the current UK Code

BS8110 Part 1⁽⁹⁾, where prescriptive rules are presented for a given fire resistance period specifying minimum cross-section sizes and nominal cover to all reinforcement. The rules are arguably conservative since they incorporate assumptions on the likely size of links and specify cover to all reinforcement, rather than the main flexural reinforcement. The prescriptive rules in Part 1 of BS8110 assume that severe spalling of the concrete will not occur if the cover is less than 40mm. In cases where the cover is greater than 40mm the designer is referred to Part 2 of BS8110⁽¹⁰⁾, where the use of steel fabric is specified as additional reinforcement. Although a reasonable solution, practically placing steel fabric between the main bars and face of the member is, at best, difficult.

BS8110, Part 2, with the sub-title '*Code of practice for special circumstances*' contains more information on the fire resistance of concrete structures. This section of the Code provides three possible design methods to determine the fire resistance, comprising tabulated data, application of test data and fire engineering calculations. In the case of tabulated data, BS8110, Part 2 removes the conservatism in the prescriptive rules given in Part 1, by specifying cover to the main flexural bars. Part 2 of BS8110 also provides limited, but important, guidance on spalling of concrete, protection against spalling and the different behaviour of aggregates in fire. However, the Code suggests that spalling of concrete is only a particular problem for reinforcement covers exceeding 40 to 50mm. The limited guidance given in the Code ignores the severe spalling that has been experienced in some fire tests⁽¹⁴⁾,⁽¹⁶⁾ and observations from real fires⁽⁴⁾ in concrete structures, where the cover to the reinforcement was significantly less than 40mm.

The ENV version⁽¹¹⁾ of the Eurocode, for the structural fire design of concrete structures (ENV1992-1-2), provides similar prescriptive rules to those given in BS8110, except that the cover to the reinforcement is specified to the centre of the bar (axis distance). The Eurocode also allows a performance-based approach to be followed where the characteristics of the fire, the thermal response and the structural performance of the member or the entire structure can be determined. The latest draft of the EN version⁽¹⁵⁾ of the Eurocode has provided some design guidance on spalling of concrete, which states that it is unlikely when the moisture content of concrete is less than 3% by weight. The Code also states that this moisture content will not be exceeded if the structure is subjected to an internal exposure. The draft EN Code also provides the guidance that if tabulated data are used, which are similar to the prescriptive rules in BS8110, then the effects of spalling can be ignored. Where spalling is considered possible, the Eurocode states that the member should be designed assuming local loss of cover to the reinforcement. However, the Code also recommends that the reduction in design strength due to spalling is not necessary for solid

slabs with evenly distributed bars or beams with a width larger than 400mm and containing more than 8 bars in the tensile area. By following the guidance given in the current draft⁽¹⁵⁾ of the EN version, the designer can ignore any detrimental effects of spalling in typical clad concrete structures.

Outside the UK, the use of high-strength concrete is starting to become popular^{(17),(18)} since it enables thinner cross-sections to be specified. However, it has been shown⁽¹⁷⁾ that this type of concrete has a high propensity for explosive spalling in fire. The latest draft⁽¹⁵⁾ of the EN version of the Eurocode has identified this problem and provides some design guidance. This guidance consists of using a supplementary steel mesh or including in the concrete mix a minimum of 2.0kg/m³ polypropylene fibres, which melt at approximately 160°C and have been shown to reduce the risk of spalling.

The full-scale concrete building at Cardington was designed for a fire resistance of 60 minutes using the prescriptive rules in BS8110, Part 1, with the addition of polypropylene fibres in the columns where high-strength concrete was used. Considering the guidance in BS8110⁽⁹⁾, ENV1992-1-2⁽¹¹⁾ and the latest EN version⁽¹⁵⁾ of the Eurocode, there should be no significant risk of severe concrete spalling or collapse of the tested building, when subjected to a realistic fire. The fire test on the building would allow the assumptions embedded in the current codified design methods to be investigated.

3. Description of the concrete building.

The full-scale seven-storey insitu-concrete building constructed at the BRE Laboratories in Cardington, was designed⁽¹⁾ to Eurocode 2 and BS8110 and represented a commercial office building situated in the centre of Bedford. The building was constructed between January and April 1998 and incorporated different concrete mixes, construction techniques and rationalisation of reinforcement layouts. The design and construction approach⁽¹⁹⁾ was developed to enable the process involved in constructing this type of building to be investigated, with the aim of producing design and best practice guidance, which ultimately increases construction economy. As a consequence, the finished building had a mixture of concrete strengths and different reinforcement layouts for each floor.

The completed building comprises 3 bays by 4 bays each 7.5m, with two core areas (Figure 1), which include steel cross-bracing to resist lateral loads. Each floor slab is nominally 250mm thick, grade C37 normal weight concrete and designed as a flat slab supported by internal columns 400mm square and external columns 400mm by 250mm. The

reinforcement in the first and second floor slabs is traditional loose bar, with hook-and-bob links for shear resistance around the columns. Above the first and second floors the reinforcement layouts change to allow different design methods and reinforcement rationalisation to be investigated.

The required funding for a large-scale compartment fire test on the concrete building was secured on the 5th July 2001. Due to the BRE corporate decision to 'mothball' the Cardington Laboratories on the 28th September 2001, the design of the test, construction of the compartment, installation of the instrumentation and execution of the test had to be carried out within a twelve-week period. This limited time-scale restricted the amount of instrumentation used and forced the test to be carried out between the ground and first floor, which saved time by removing the need to transport personnel and materials up the building.

A fire compartment, with a floor area of 225m², was constructed between the ground and first floors (Figures 1 and 2). Due to the height of the footings, the overall height of the compartment from the laboratory floor to the underside of the first floor was 4.25m (Figure 2). The compartment walls were constructed using 140mm thick Topcrete blocks and lined, inside the compartment, with one skin of plasterboard. The laboratory floor was also lined with plasterboard. The blockwork was constructed such that a gap of 325mm was left between the top of the wall and underside of the slab. This gap was filled with a ceramic blanket which was fixed to the wall and shot fired to the underside of the slab (Figure 3). It was assumed that the design and construction of the compartment wall would contain the fire, whilst allowing vertical movement of the tested structure, without providing any support.

The tested first floor slab was nominally 250mm thick and designed as a flat slab. The reinforcement comprised 12mm and 16mm loose bars at various centres. 12mm diameter hook-and-bob links were provided around the columns consisting of 72 links around the internal columns, 60 links around an external column and 41 links around a corner column. The floor was constructed using a C37 normal-weight concrete, with a cube strength at 28 days of 61N/mm². Tests on the wet properties of the concrete using a Rapid Analysis Machine (RAM) showed that the average cement content was 407 kg/m³ and the average water content was 205 kg/m³. The aggregate source was Bardon Little Paxton and was composed predominantly of flint (chert) material, which, as previously discussed, makes the concrete susceptible to aggregate spalling. The average moisture content, measured seven days before the fire test, was 3.8% by weight and the average measured permeability 48mm into the floor slab was $6.75 \times 10^{-17} \text{ m}^2$.

One internal column was fully exposed to the fire and 8 columns (internal and external) were partially exposed to the fire (Figure 1). All columns were Grade C85 high-strength concrete and contained limestone aggregate and micro silica. To reduce spalling in fire, 2.7kg/m³ of polypropylene fibres were added to the concrete mix. This dosage of fibres was higher than the current minimum value of 2.0kg/m³ specified in the latest EN draft⁽¹⁵⁾ version of the Eurocode. However, at the time of casting the columns, the draft Eurocode did not provide any recommendations and the designers of the building had to estimate a required dosage based on the limited data available. At 28 days the average measured cube strength was 103 N/mm². The average moisture content, measured seven days before the fire test, was 4.2% by weight and the average measured permeability 48mm into the column was $1.92 \times 10^{-19} \text{ m}^2$.

The design specification for the building stated that the nominal cover to all reinforcement should be 20mm. To check if this cover was achieved for the completed structure, a cover survey was carried out prior to the test, with the results shown in Table 1. This survey showed that the cover to the reinforcement was more than sufficient to achieve at least 60 minutes fire resistance, as specified in current codes^{(9),(11)}. The codes also state a minimum size for members to achieve a required fire resistance. For 60 minutes BS8110 Part 1 gives a minimum slab thickness of 95mm and a minimum column width of 200mm, for columns fully exposed to the fire. The section sizes on the Cardington building were significantly greater than these values and therefore had a greater fire resistance than the minimum specified.

Structural Member (Area)	Cover to main reinforcement (mm)
First Floor Slab: gridlines B to C and 2 to 3.	22
First Floor Slab: gridlines B to C and 3 to 4	25
First Floor Slab: gridlines C to D and 2 to 3	24
First Floor Slab: gridlines C to D and 3 to 4	22
Columns D2, D3, D4: ground to first floor	42
Columns B2, B3, B4, C2, C3, C4: ground to first floor.	40

Table 1: Measured cover to main reinforcement.

The design load for the floors is shown in Table 2. The behaviour of structures subjected to fire can be treated as an accidental limit state, with appropriate load factors. These factors are shown in Table 2 and are based on the values given in ENV 1991-1⁽²⁰⁾ and the proposed amendment⁽²¹⁾ to BS5950 Part 8⁽²²⁾.

Type of design load	Design load (kN/m ²)	Partial load factor at fire limit state	Design load at the fire limit state (kN/m ²)
Self weight (assuming density of 24kN/m ³)	6.0	1.0	6.00
Raised Floor	0.5	1.0	0.50
Ceiling and services	0.5	1.0	0.50
Imposed load (BS6399-1)	2.5	0.5	1.25
Partitions	1.0	1.0	1.00
Total design load at the fire limit state			9.25

Table 2 Design load at the fire limit state.

To represent the design imposed, partition, raised floor, ceiling and services load, sand bags each weighing 10.79kN, were placed on the first floor to achieve a vertical load of 3.25kN/m² over the fire compartment area. Additional sandbags were placed on the floors above the tested first floor to achieve a total design axial load, excluding self-weight, of 925kN in column C3 (central column within the fire compartment area) and 463kN in column D3. This design load assumes a 40% reduction in the partition and imposed load as stated in BS6399 Part 1⁽²³⁾.

A protected steel safety cage, comprising four steel columns, which were tied together, was placed around column C3 within the fire compartment. The cage was designed to support the first floor slab should the tested 400mm × 400mm high-strength concrete column (C3) fail during the fire. The floors above the first floor were shown, by simple calculation, to be adequate to bridge column C3 by tensile membrane action. The constructed cage is shown in Figure 4. The cage was not required during the test since the concrete column performed adequately, as discussed in Section 6.0.

Instrumentation was installed in the tested area to measure horizontal and vertical displacements, atmosphere temperatures, concrete surface temperatures, the distribution of temperature through the slab and surface strains on the first floor surface.

4. Design of the fire

The design of the fire was based on the parametric approach from Annex A of the fire part of the Eurocode for Actions⁽²⁴⁾. An attempt was made to simulate, as closely as possible, realistic conditions in terms of fire loading, thermal properties of the compartment linings and ventilation conditions.

A number of issues complicated the calculation of the predicted compartment time-temperature response, which include:

- The compartment height was 4.25m. This is outside the scope of the codified calculation procedure. The majority, if not the entirety, of full-scale tests on which the calculations are based and the method validated have compartment heights less than or equal to 4m.
- In a realistic compartment the soffit would not be exposed. The reinforced concrete ceiling to the compartment has a high thermal inertia. This effectively reduces the rate of temperature rise within the compartment as heat is absorbed into the ceiling rather than radiated back into the compartment.
- Optimum ventilation conditions to achieve flashover are obtained by limiting the available ventilation in the early stages of the fire. This would have required a moveable screen to achieve the required optimum conditions. The provision of such a screen was deemed unacceptable as being unrealistic and unsuitable for validating design methods.

A number of different design fire scenarios were considered. The first scenario predicted low peak temperatures (840°C) with a long duration (in excess of 100 minutes). The second scenario predicted high maximum temperatures (in excess of 1000°C) but had a short duration of approximately 30 minutes. It is a matter of some debate as to which is the most severe scenario for this form of construction. For example, a high temperature, short duration fire, may induce concrete spalling due to the thermal shock, whereas a lower temperature but longer duration will result in a greater average temperature in the concrete members.

It was decided that a realistic office fire would be obtained by providing ventilation to the compartment from full-height openings, creating a total area of 27.2m², along gridline D, which represents an opening that is equivalent to 12% of the compartment floor area. The fire consisted of timber cribs creating a fire load of 40kg/m² (720MJ/m²). The walls and floor of the compartment were lined with one skin of plasterboard. Figure 4 shows the fire compartment prior to ignition. The predicted time-temperature response from the Eurocode is shown in Figure 5.

5. Test results and observations.

The test was conducted on the 26th September 2001 (Figure 6) and was witnessed by a number of experts from the UK and Europe. The instrumentation was checked before the fire test and shown to be working satisfactorily. Unfortunately, all the instruments malfunctioned approximately 18 minutes after ignition. The following explanation of the malfunction of the instruments is presented to aid future researchers carrying out similar large-scale tests and to

highlight to practitioners the possible weakness of relying on fixings into the soffit of slabs during a fire, especially where fixings may provide restraint to compartment walls.

All the electrical cables to the instruments were run horizontally above the first floor to one point and then bound and dropped vertically at column A3 (Figure 1). These vertical cables were 7.5m from the fire compartment and were unprotected, since during the planning of the test they were considered to be a sufficient distance from the fire area. The compartment wall comprised blockwork, plasterboard and a ceramic blanket, as discussed in Section 4.0. The blanket was shot fired to the soffit of the first floor. However, when the concrete to the soffit began to spall, the fixings to the blanket failed. The fire escaped the compartment through the top 325mm opening and travelled horizontally 7.5m and burnt through the cables (Figure 7). The loss of the fixings in the soffit of the slab is shown in Figure 8. Where spalling did not occur the fixings remained in place, as shown in Figure 9. Any future design of compartment walls should consider the possibility of spalling of the concrete causing the loss of any fixings into the slab's soffit, which may lead to compartmentation failure.

The maximum atmosphere temperature, recorded during the test near to column B3, before malfunction of the instruments was 950°C. Figure 10 shows the recorded maximum atmosphere temperature and the average atmosphere temperature throughout the fire compartment. Considering Figure 10, it can be seen that the temperatures reduced considerably between 12 and 13 minutes after the start of logging (6 minutes after ignition). A video and time-lapse photos of the test show clearly that explosive spalling of the soffit of the floor slab began at this time. The spalling, which was extensive, reduced the severity of the fire throughout the test, which is an effect that has also been shown from other fire tests⁽²⁵⁾.

The vertical displacements of the slab, at the locations shown in Figure 1, are shown in Figure 11, up to the time at which the instruments malfunctioned. It can be seen that the displacements near the centre of the building, at locations V4, V5, V8, V11 and V12 show a distinct plateau of constant displacement after 15 minutes. The displacements at locations V6, V9 and V13, towards the edge of the building, are much larger than the displacements near the centre and show no signs of a plateau.

The failure of the blanket around the top of the compartment increased the ventilation to the fire. Numerical models^{(12),(18),(24)} used to predict the time-temperature response suggest that this increase in ventilation had the effect of slightly increasing the maximum temperature of fire and shortening the duration of the fire.

6. Damage to the structure.

The structure showed no signs of collapse during or after the fire. Following the fire, the displacement transducers were re-wired and the residual displacements recorded. The residual vertical and horizontal displacements at first floor level are shown in Figure 12. The extent of the horizontal movement of the columns along gridline D, caused by thermal expansion of the heated slab, can be appreciated from the picture shown in Figure 13. The horizontal movement of the floor slab also caused buckling of the steel cross-bracing positioned on gridline B and C (refer Figure 12), as shown in Figure 14. An obvious question is why is there a lateral residual movement of 67mm of column D3, and why did it not recover as the slab cooled and the thermal expansion reduced? The column itself had insufficient strength to hold the slab out and no significant tensile cracks were observed in the floor slab. The only plausible explanation, at present, is that when the slab is heated the aggregate expands and the cement paste contracts. The difference in expansion causes internal cracking between the aggregate and paste resulting in overall irrecoverable thermal expansion. In addition the phase changes of siliceous aggregates when heated produce an increase in volume that is irreversible on cooling⁽⁴⁾.

There was significant cracking around column D3 (Figure 15), which can be attributed to the hogging moment from the slab and the moment caused by the P- δ effect from the vertical load in the column and its lateral movement due to thermal expansion of the floor slab. The behaviour of the structure and recorded crack layout can be investigated more fully by the planned computer modelling of the test.

The extent of the spalling, which exposed the bottom reinforcement in the heated floor slab is shown in Figure 16. The pictures shown in Figures 17 and 18 also provide an appreciation of the extent of the spalling, and show reinforcement hanging from the soffit of the floor slab. A total of 10 reinforcement bars, 12mm diameter and 2.6m long were found on the compartment floor.

Examination of the area of the spalling, and the type of explosive nature of the spalling observed during the test, suggests that high compressive forces were induced in the slab due to restraint to thermal expansion. The behaviour of the spalling around the service holes (Figure 19) also suggests that spalling was primarily due to high compressive stresses, since where the compressive stresses are reduced along the edges of the hole no spalling occurred. Although these compressive stresses almost certainly led to the severity of the spalling, they

were also probably beneficial to the survival of the floor slab due to compressive membrane action. Provided the slab does not reach vertical displacements greater than approximately 0.5 times its depth⁽²⁶⁾, then the effect of compressive membrane action will considerably enhance the load-carrying capacity of the slab above that calculated assuming flexural action. The behaviour of compressive membrane action will be considered in more detail during the planned numerical modelling of the test.

The debris from the spalling on the floor of the compartment (Figure 20) was 90mm deep at the worst location. This debris clearly reduced the severity of the fire, as observed during the test.

All the columns, which were high-strength concrete, behaved well in the fire with no signs of surface spalling, suggesting that the polypropylene fibres, which would have melted at approximately 160°C, alleviated the pore pressures. However, it should be noted that the dosage of fibres used in the tested columns was higher than the current minimum value given in the Eurocode, which unfortunately means that the fire test is unable to validate the current Eurocode recommendations.

Inspection of the columns after the test showed that longitudinal cracks, approximately 50mm into the column had formed (Figure 21). It was relatively easy to remove the outer concrete as shown in Figure 22. The behaviour of the column shown in Figures 21 and 22 is typical of corner spalling.

7. Discussion and Conclusions

The main observations from the large compartment fire test on the full-scale concrete building have been presented in this paper. It is worth emphasising the limitations of the test in that it only represents one type of construction and one type of fire scenario. In addition, instrumentation failure during the test, undoubtedly, hinders our understanding of the structural behaviour and highlights the risks in conducting a ‘one-off’ large-scale fire experiment. However, the limited results obtained during the test, the measured residual displacements, and the observed damage, provide useful information on the holistic behaviour of concrete buildings in fire.

The current codes, and forthcoming EN version of the Eurocode, suggest that by using the prescriptive rules, consisting of tabulated data, concrete spalling of the Cardington concrete building should not be significant during a fire and could be ignored in design. If the

designer adopts a performance-based approach then the latest EN version of the Eurocode states that spalling is unlikely when the moisture content is less than 3%. Although the measured moisture content in the tested floor slab on the Cardington building was 3.8%, in practice, the moisture content of the concrete prior to a fire would be unknown. The Eurocode recognises that the designer will generally not know the moisture content and provides recommendations which suggest that the floor slab in the Cardington building is unlikely to spall in fire, since it is subject to an internal exposure. In addition, the Eurocode also states that if the concrete is likely to spall (i.e. the designer feels that the moisture content is greater than 3%) its design strength is unaffected for solid slabs with evenly distributed bars. Therefore, irrespective of the design approach used, the codes advise the designer to ignore the effects of spalling for the design of the slab.

The fire test showed that spalling to the soffit of the slab was extensive, exposing the bottom reinforcing bars. The spalling, which was explosive, was almost certainly due to the high in-plane compressive stresses in the slab, caused by the restraint to thermal expansion and the high pore water pressures⁽⁷⁾. Although the bottom bars were totally exposed to the fire, and thus the flexural capacity of the slab was drastically reduced, the slab remained stable and supported the applied static load by compressive membrane action. Considering this behaviour it could be argued that the codified approach of ignoring spalling is correct, since the slab continued to support the load. However, if the designer is ignoring the effects of spalling and relying on compressive membrane action then the mechanics of the behaviour should be understood and its limits defined. Compressive membrane action can only occur at small displacements (typically displacements less than half the thickness of the slab) and can occur in flat slabs, as shown from numerous previous tests^{(27),(28),(29)}. At large displacements tensile membrane action occurs, but its effects are small in flat slabs where the only vertical support is at the columns. For the slab to be supported by tensile membrane action, some vertical 'line' support between columns is required⁽³⁰⁾. This approach has been used successfully in the fire design of concrete composite floors in steel framed buildings^{(31),(32)}. However, it should be noted that for tensile membrane action, the reinforcing bars support the load and therefore need to retain their strength during a fire, which can only be achieved if sufficient concrete cover to the bars is maintained. During the test on the concrete building, the slab's vertical displacement was small and the static load was supported due to compressive membrane action. However, if the slab's vertical displacements were greater, due to perhaps a more severe fire, or a larger compartment area such that there was no cooler structure and thus lateral restraint surrounding the heated slab, then it is difficult to see how the slab could have supported the static load.

Designers should also consider the spalling of concrete when specifying fixings between compartment walls and the soffit of concrete slabs. One of the primary aims of structural fire design is to ensure that the fire remains in the compartment of origin. If fixings into the concrete structure are specified to provide restraint or support to the compartment wall then these fixings should remain in place if spalling occurs. The effect of concrete spalling on fixings was not foreseen in the fire test discussed in this paper, with the consequence of the fire escaping the compartment area and destroying the cables, resulting in the loss of valuable data.

The lateral movement of the heated slab in the test was significant, resulting in buckling of the steel cross-bracing and lateral displacement of the external columns. The buckling of the steel cross-bracing reduced the restraint slightly to the heated slab, whereas if concrete shear walls were used instead, the restraint to thermal expansion would have been greater, inducing higher compressive stresses in the slab and possibly increasing the severity of the spalling. The lateral movement of the external columns has been identified as the cause of previous failures⁽⁵⁾ of concrete structures, although the test discussed in this paper showed that the Cardington building was able to accommodate these movements. However, the designer should be aware of this behaviour and ensure that the columns can withstand the likely lateral displacement during the fire.

Some readers may be tempted to compare the behaviour of the test on the concrete framed building with the six fire tests^{(33),(34)} carried out on the steel-framed building at Cardington. However, this comparison is not straightforward since two of the natural fire tests on the steel framed building had a higher fire load (45kg/m^2) and in one test, real furniture instead of wooden cribs was used. In addition, the thermal characteristic of the floor slab is different in the separate buildings. In the case of the steel-framed building the steel deck to the composite slab will radiate more heat back to the fire, whereas in the concrete building the concrete soffit will absorb more heat from the fire. This will have the effect of higher atmosphere temperatures in the steel building compared to a similar fire in the concrete building. From a structural point of view, the floor slab in the steel building was designed to have 90 minutes fire resistance, whereas the slab in the concrete building was designed to have 60 minutes. Finally, the applied static load, excluding self-weight, on the concrete building was higher than the load applied during all the six tests on the steel-framed building. Therefore, although it is tempting to compare the results from the fire tests on different forms of construction, it can be misleading. The way forward, however, is to understand fully the behaviour of entire structures constructed from different materials and produce design guidance based on this behaviour, which will ultimately lead to the construction of safer,

more economical, buildings. This process has significantly progressed for steel-framed structures with composite slabs^{(30),(32)}. The same analytical research work is now required for concrete structures, and perhaps then a true comparison between different construction materials in fire can be made.

Acknowledgements

Funding for the fire test described in this paper was provided by British Cement Association, Building Research Establishment, Reinforced Concrete Council, CONSTRUCT, Cembureau, Febelcem, Tarmac and Unifrax. The support from these organisations is gratefully acknowledged. The views expressed in this paper are those of the Author and may not necessarily be those of the funding bodies.

References

1. Chana P. The European concrete building project. *The Structural Engineer*. Vol 78/No.2 Jan 2000. pp 12-13.
2. Papaioannou K. The Conflagration of Two Large Department Stories in the Centre of Athens. *Fire and Materials*. Vol. **10**, 1986. pp 171-177.
3. Malhotra H.L. Some noteworthy fires in concrete structures. Paper presented at the 8th FIP Congress, held 30 April to 5 May 1978 pp 86-98. Borehamwood, FRS 1978.
4. *Fire resistance of concrete structures*. Report of a Joint Committee of the Institution of Structural Engineers and the Concrete Society. The Institution of Structural Engineers. 1975.
5. Van Acker A. Fire design of concrete building structures: Needs and priorities from a designers point of view. Workshop: Fire design of concrete structures. Malta 2001. Organised by Task Group 4.3 of the International Federation for Structural Concrete (ed. Niels Peter Høj)
6. Concrete for fire resistant construction. A Cembureau report prepared with the assistance of FIP. Cement and Concrete Association 1979.
7. Khoury G.A. Effect of fire on concrete and concrete structures. *Progress in Structural Engineering and Materials*. 2000 **2**: pp 429-447.
8. Malhotra H.L. Spalling of Concrete in Fires. CIRIA Technical Note 118 1984.
9. BS8110: Part 1: 1989: Structural use of concrete. Part 1: Code of practice for design and construction. British Standards Institution, London.
10. BS8110: Part 2: 1985: Structural use of concrete. Part 2: Code of practice for special circumstances. British Standards Institution, London
11. DD ENV 1992-1-2:1996. Eurocode 2: Design of concrete structures: Part 1.2 General rules – Structural fire design (together with United Kingdom National Application Document) British Standards Institution, London.

12. Purkiss J.A. *Fire Safety Engineering Design of Structures*. Butterworth 1996.
13. Connolly R.J. *The spalling of concrete in fires*. PhD Thesis. Aston University 1995.
14. Davey N. and Ashton L.A. *National Building Studies Research Paper No. 12: Investigations on Building Fires Part V. Fire Tests on Structural Elements*. HMSO 1953.
15. prEN 1992-1-2 (2nd Draft) Eurocode 2: Design of concrete structures – Part 1.2: General rules – Structural fire design. European Committee for Standardisation, Brussels. July 2001.
16. Purkiss J.A., Morris, A.W. and Connolly R.J. Fire resistance of reinforced concrete columns – correlation of analytical methods with observed experimental behaviour. *Interflam 96*, (eds. Franks C.A. and Grayson S.), Interscience Publications Ltd., London. pp 531-541.
17. Kodur V.K.R. Studies on the fire resistance of high-strength concrete at the National Research Council of Canada. *Proceedings of the International Workshop on Fire Performance of High Strength Concrete*. pp 75-86. NIST Special Publication 919. National Institute of Standards and Technology, USA.
18. Buchanan A.H. *Structural Design for Fire Safety*. John Wiley & Sons Ltd 2001.
19. Chana P. and Moss R.M. Re-engineering the design and construction processes for in situ concrete frames. *Structural Concrete*. 2001. 2 No. 2 June. pp 49-61.
20. DDENV 1991-1: 1996. Basis of design and actions on structures - Basis of design (together with United Kingdom national application document). British Standards Institution, London.
21. Bailey C.G. and Brettle M. *Background Document: Amendment to BS5950-8: Partial load factors on imposed loads for fire design*. Internal BRE Report. Building Research Establishment, Garston. 2001.
22. BS5950 Part 8: 1990. Structural use of steelwork in buildings: Part 8: Code of practice for fire resistant design. British Standards Institution, London. 1990.
23. BS6399 Part 1: 1996. Loading for buildings Part 1. Code of practice for dead and imposed loads. British Standards Institution, London. 1996.
24. prEN1991-1-2: Eurocode 1- Action on Structures. Part 1.2: General Actions – Actions on structures exposed to fire. Final Draft (Stage 49) 19 December 2001. European Committee for Standardisation, Brussels..
25. Natural Fire Safety Concept, Full scale tests, implementation in the Eurocodes and development of a userfriendly design tool, Technical report No 5, Profil Arbed Recherches, March 2000.
26. Park R., Ultimate strength of rectangular concrete slabs under short-term uniform loading with edges restrained against lateral movement. *Proceedings, Institution of Civil Engineers*, 28, pp 125-150.
27. European Committee for Concrete. Information Bulletin No. 35. *The application of the yield-line theory to calculations of the flexural strength of slabs and flat-slab floors*. Cement and Concrete Association, London. 1962.
28. Hatcher D, S, Sozen M. A., and Chester S.P., Test of a reinforced concrete flat slab. *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers*. ST6 June 1969. pp 1051-1072.
29. Blakey F.A., Australian experiments with flat plates. *Jour. of American Concrete Institute*. April 1963. pp. 515-525.
30. Bailey C.G. Steel structures supporting composite floor slabs: design for fire. BRE Digest 462. Building Research Establishment. 2001

31. Bailey C.G and Moore D.B. The structural behaviour of steel frames with composite floorslabs subject to fire: Part 1: Theory. *The Structural Engineer*. Vol. 78/No.11 June 2000.
32. Newman G.M, Robinson J.T and Bailey C.G. *Fire Safe design: A New Approach to Multi-Storey Steel-Framed Buildings*. The Steel Construction Institute 2000.
33. Bailey, C. G., Lennon, T. and Moore, D. B., The behaviour of full-scale steel framed buildings subjected to compartment fires, *The Structural Engineer*, Vol. 77, No. 8, April 1999. pp. 15-21.
34. Martin, D. M., and Moore, D. B., Introduction and background to the research programme and major fire tests at BRE Cardington, National Steel Construction Conference, London, 13th-14th May 1997. pp. 37-64

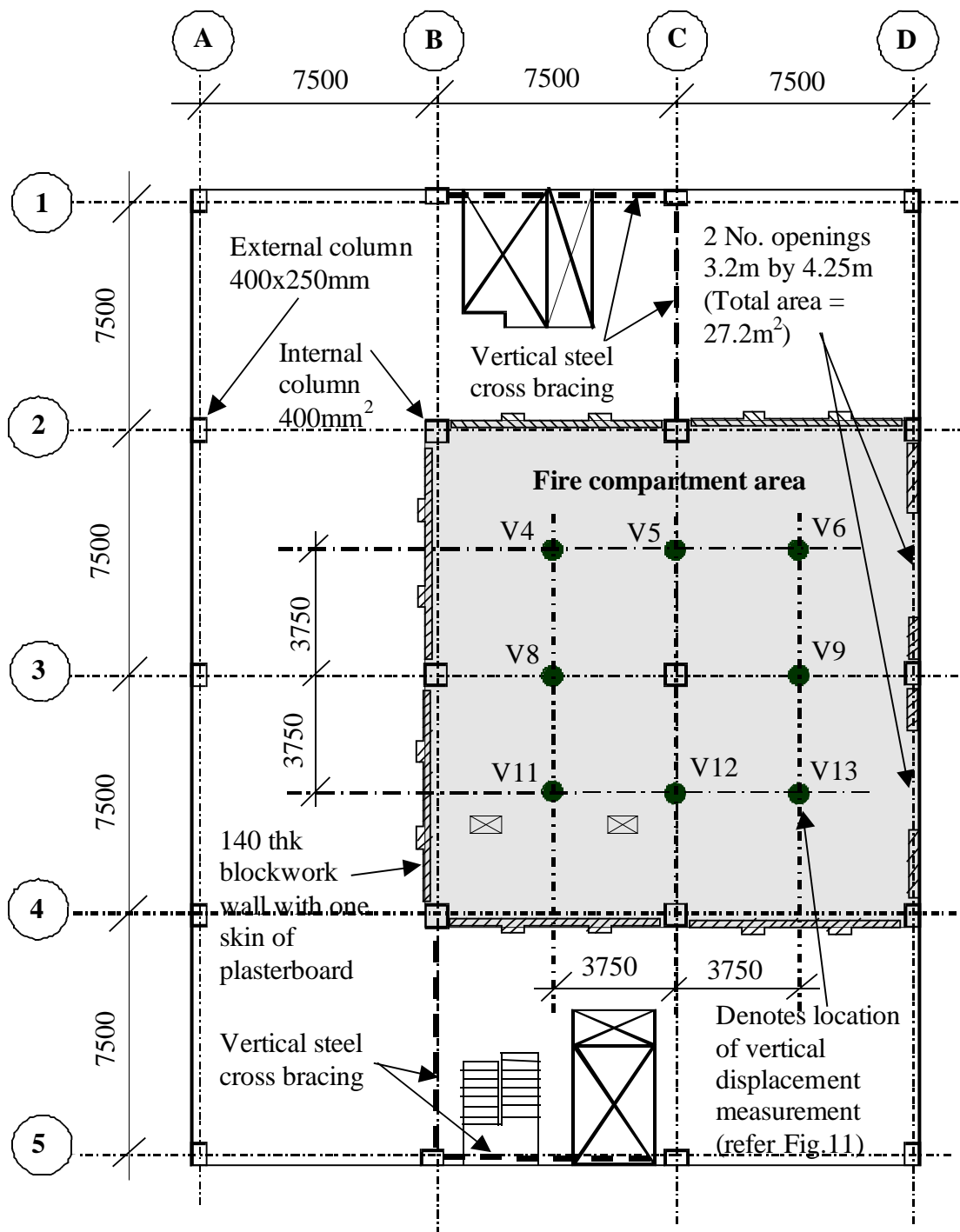


Figure 1: Plan of building showing location of fire compartment.

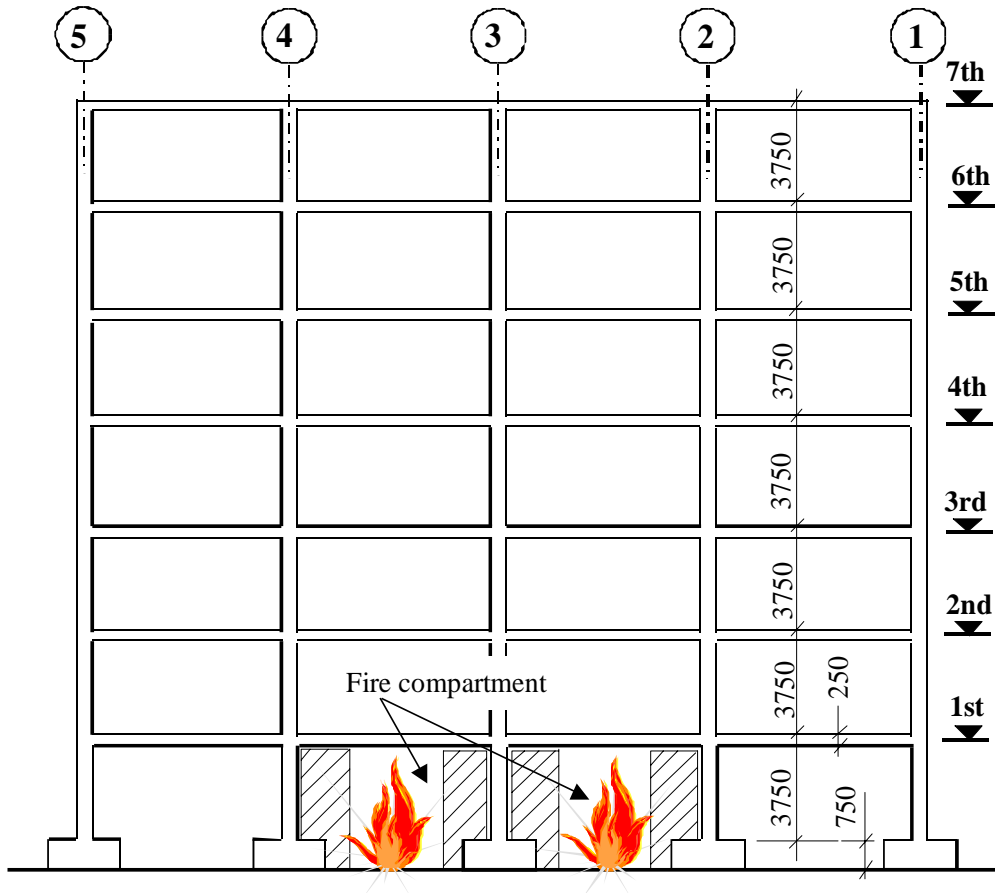


Figure 2 Cross-section through the building showing location of fire test.

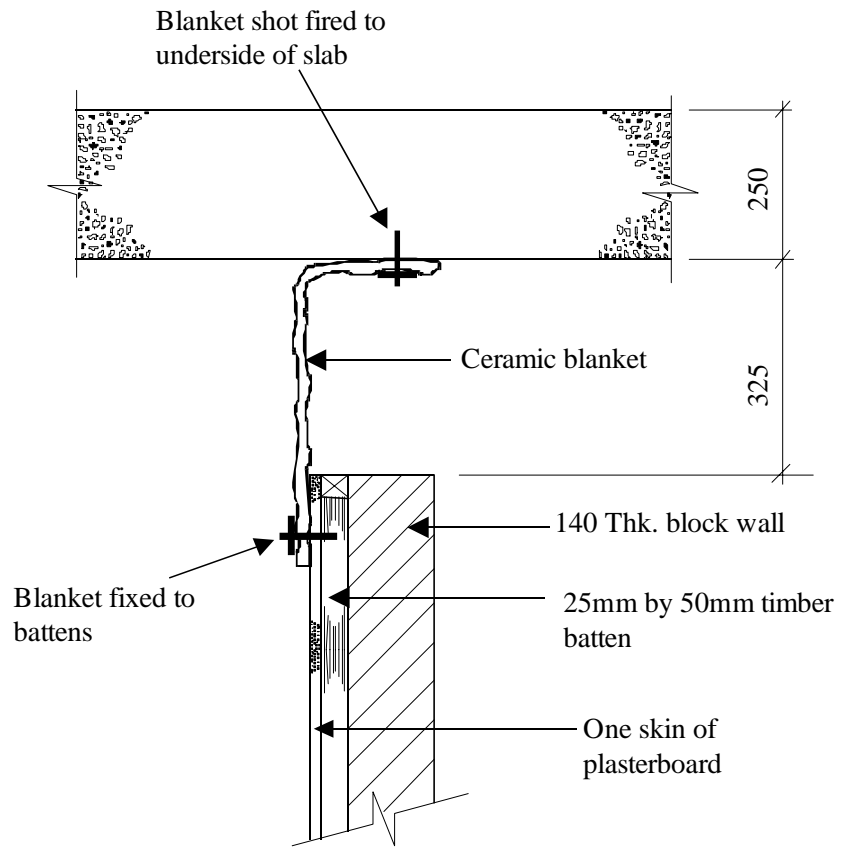


Figure 3 Detail showing fixing of ceramic blanket to first floor soffit and compartment wall.



Figure 4 Fire compartment prior to ignition (looking from the openings along gridline D)

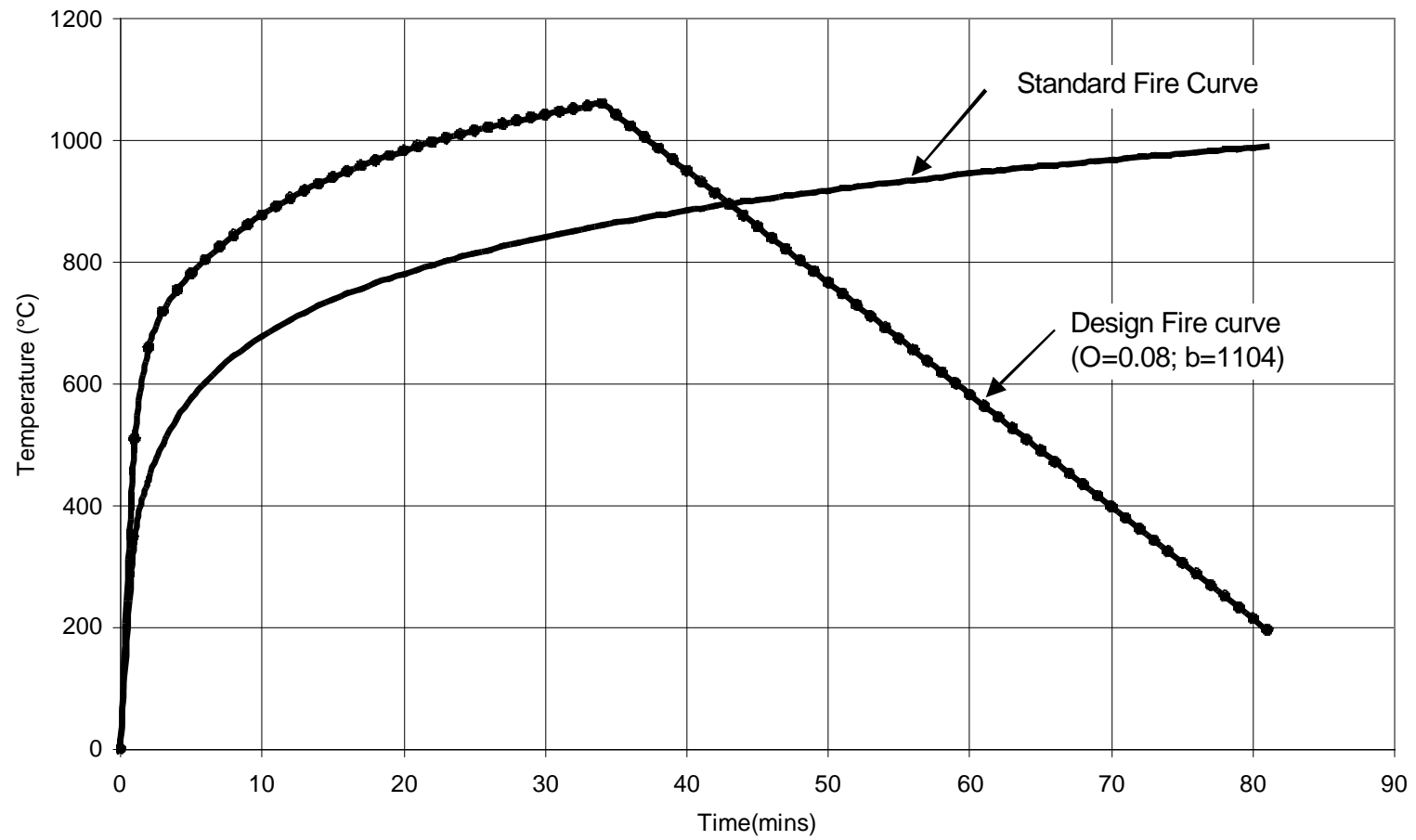


Figure 5 Design fire curve and standard fire curve.



Figure 6 Fire test (Note the start of the spalling of the concrete soffit)

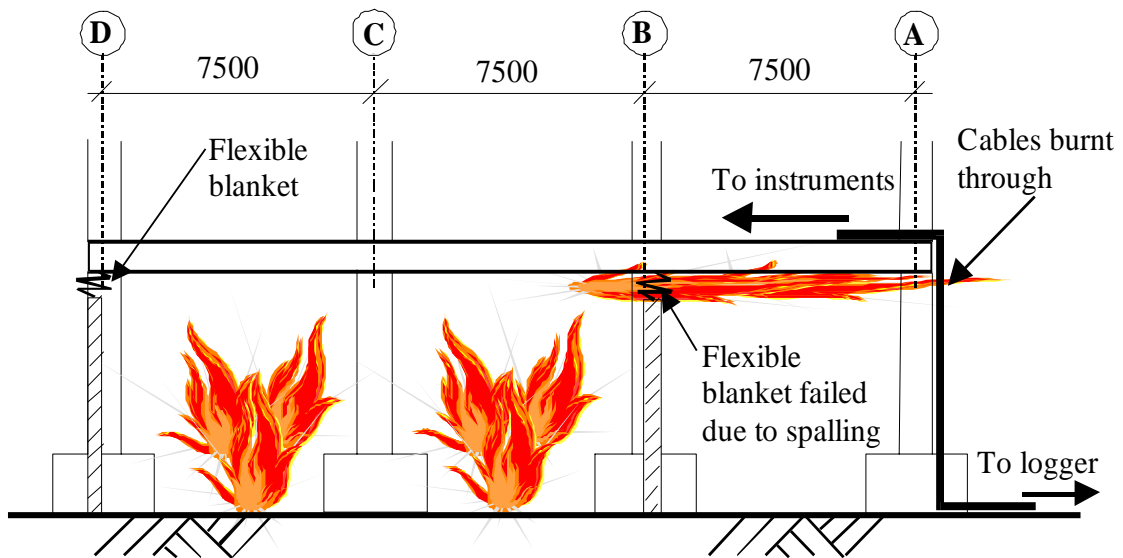


Figure 7: Diagram showing cause of instrumentation malfunction.



Figure 8 Picture looking at column C2 showing spalling to soffit of slab
(Note: Failure of blanket fixings into slab soffit)



Figure 9 Picture looking at column D3
(Note: Fixings between blanket and soffit are intact due to limited spalling)

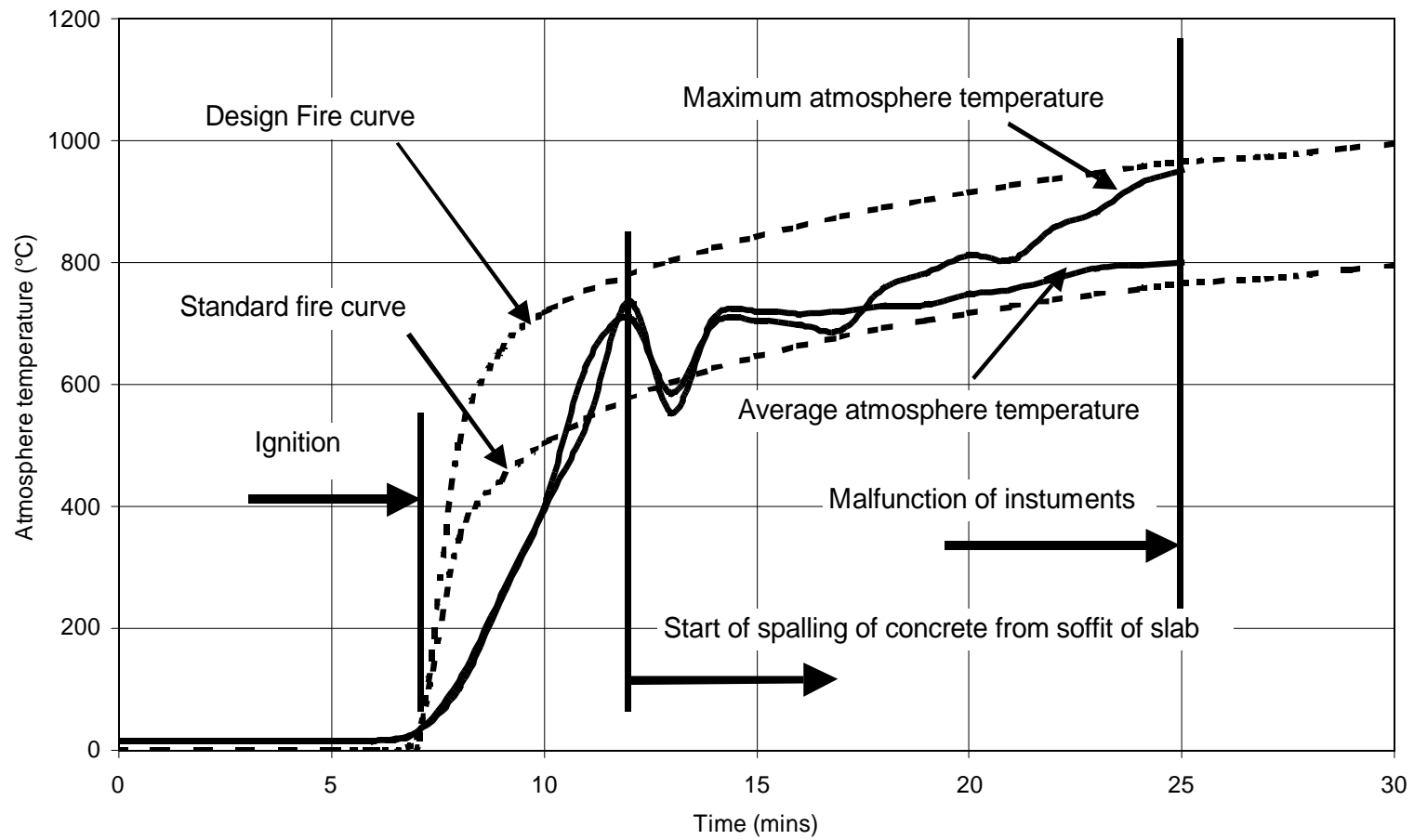


Figure 10 Recorded atmosphere temperatures throughout the compartment 300mm below the soffit of the slab.

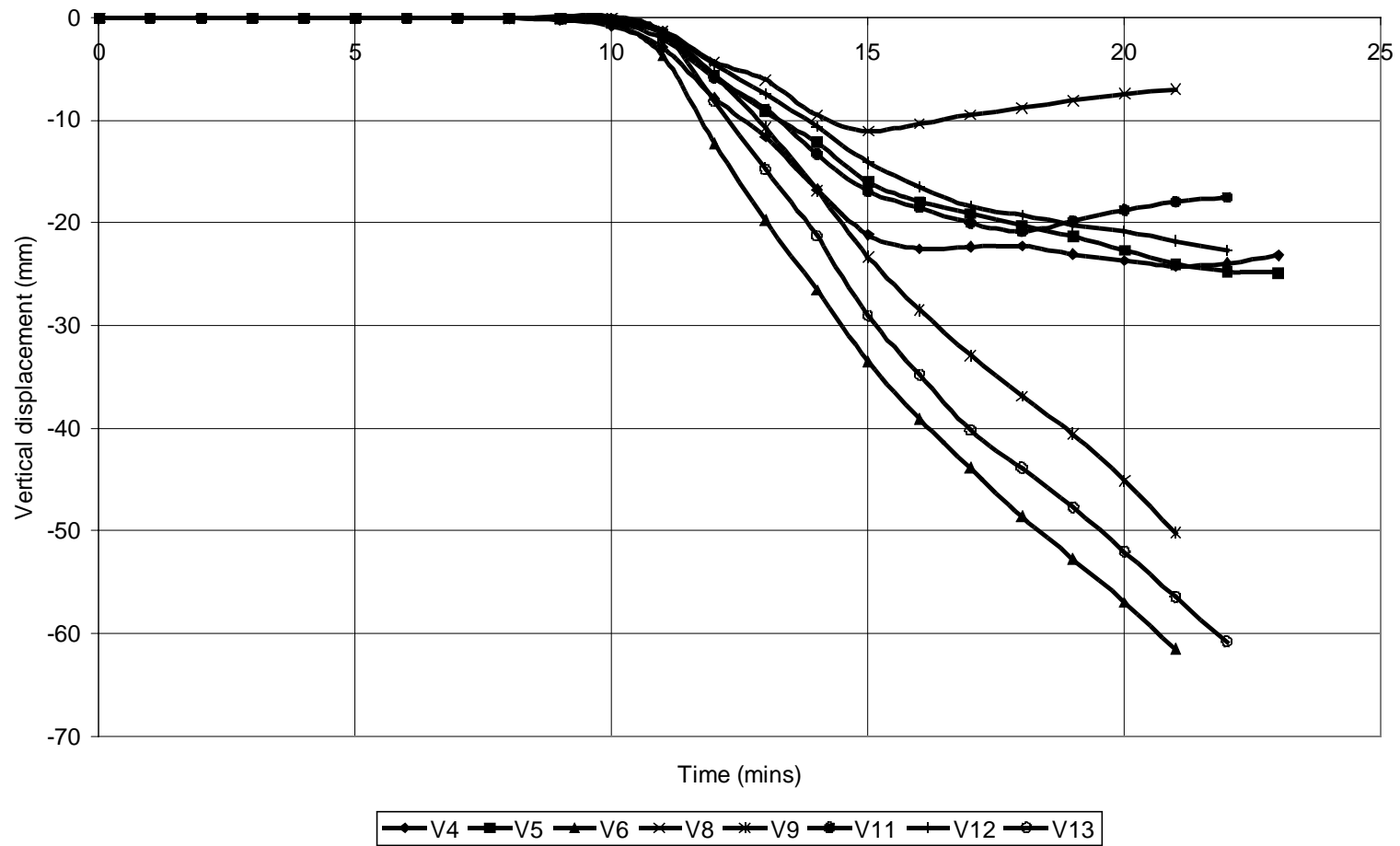


Figure 11 Measured displacements of the slab.
 (For location of measurements refer Figure 1)

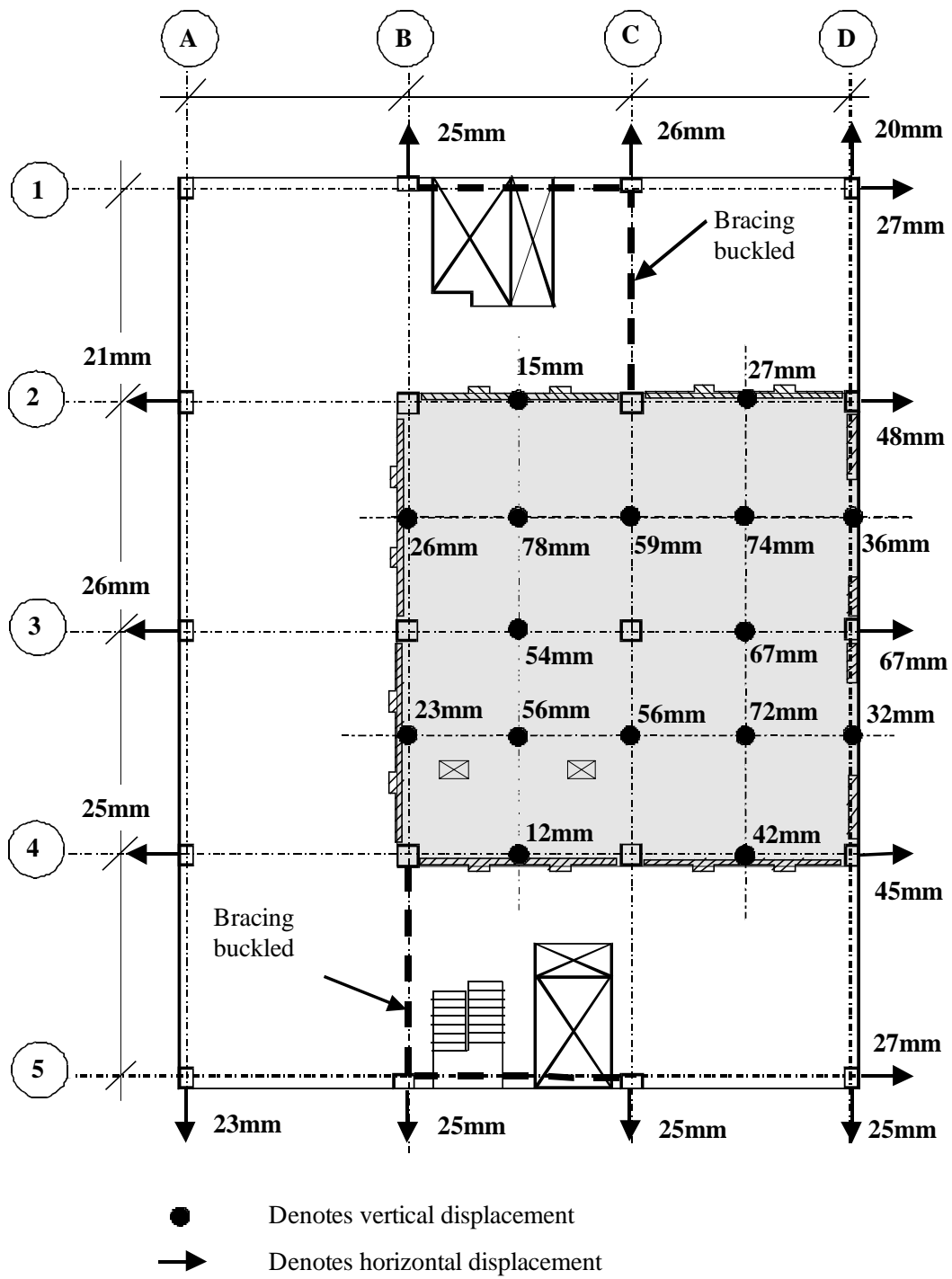


Figure 12 Residual horizontal and vertical displacements



Figure 13 Picture showing lateral displacement of external columns.

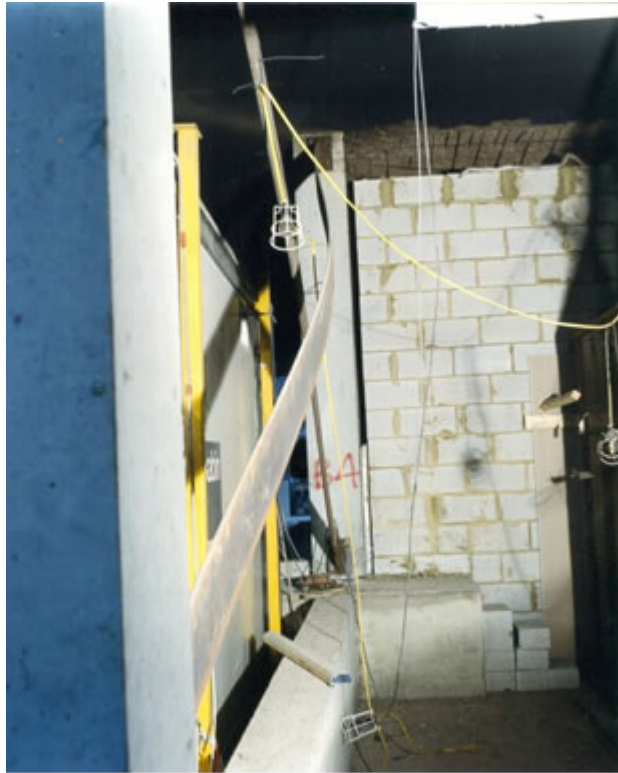


Figure 14 Buckling of bracing between columns B4 and B5 (similar buckling of bracing between columns C2 and C1 was also observed).



Figure 15 Cracking in the proximity of column D3

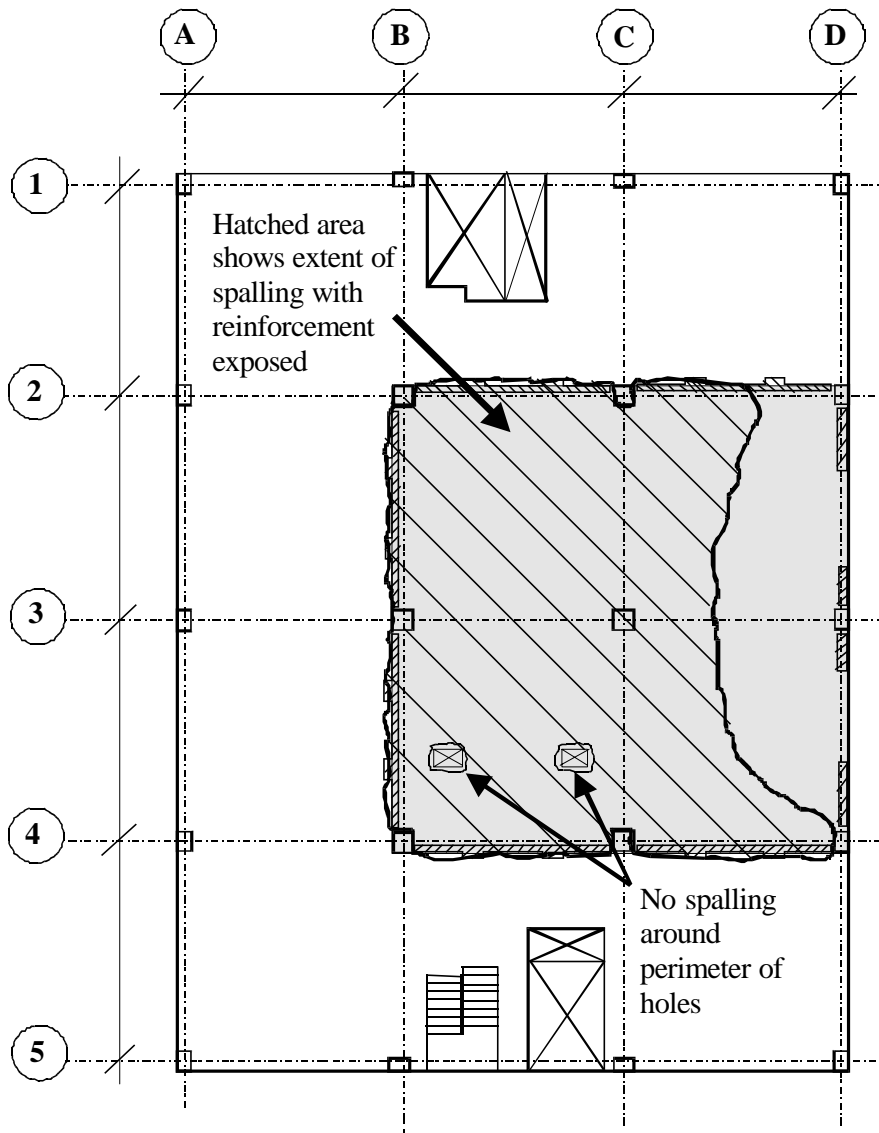


Figure 16 Extent of the spalling exposing bottom reinforcement in the slab.



Figure 17 Picture looking into compartment from column D2 showing extent of spalling.



Figure 18 Picture showing spalling (looking at column B2)



Figure 19 Picture showing spalling around service hole



Figure 20 Picture showing extent of spalling (debris) on compartment floor
(50 pence shown for scale)



Figure 21 Longitudinal cracks in Column C3.



Figure 22 Removal of outer (corner) concrete on column C3