STRUCTURAL FIRE ENGINEERING

INVESTIGATION OF BROADGATE PHASE 8 FIRE

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Investigation of Broadgate Phase 8 Fire
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EXECUTIVE SUMMARY

This report results from an investigation of the performance of a steel framed building after a serious fire. The report and investigation were initiated by the developer Stanhope Properties PLC, and the industry corporate body SCIF (Steel Construction Industry Forum). SCIF is the result of a co-operation by British Steel, the British Constructional Steelwork Association, and The Steel Construction Institute. These two organisations brought together a diverse and highly experienced body of professional engineers and managers to study the fire and its structural effects. The report is unique in that it is the first in depth study, calling upon such a wealth of expertise, of a modern steel framed building subject to a severe fire.

On the 23rd June, 1990 a fire developed in the partly completed fourteen storey building in the Broadgate development.

Fire began in a large contractor's hut on the first floor and smoke spread unchecked throughout the building. The automatic fire detection and alarm system or sprinkler system was not yet operational when the fire occurred.

The total duration of the fire was in excess of 4 1/2 hours of which 2 hours represented the most severe phase of the fire with temperatures in excess of 1000^{o}C. The direct fire loss was in excess of £25M, of which less than £2M represented structural frame and floor damage; the balance in the main arising from smoke damage to the building fabric.

The structure of the building was a steel frame with composite steel deck/concrete floors. The steel structure was partially unprotected at this stage of the construction. The fire caused shortening of several unprotected columns of about 100mm due to plastic deformation and large deflections in some composite steel beams. Despite the large deflections, the structure behaved well and there was no collapse of any of the columns, beams or floors.

This fire proved that the design of the connections of the floor slabs to the beams and of the beams to the columns were able to accommodate large deformations and forces without failure.

The fact that the building was constructed of steel framework and composite floors meant that replacement of damaged members could be undertaken in a very short time - the structural repairs took only 30 days to complete.

The behaviour of the structure and the floor members showed that steel frames when designed in accordance with BS 5950 : Part 8 - were safe under severe fire exposure conditions.

This report examines the fire and analyses the structural behaviour. The conclusions lend support to the philosophy behind the new Code and the Draft Approved Document B of the Building Regulations. The recommendations give direction to reducing the risk of further such fires occurring and to the formulation of more economic and safe steel structures and associated fire protection.

This study shows that when fire affects only part of a structure and when the framework acts as a total entity, which is typical of multi-storey buildings, structural stability is significantly improved.

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CONTENTS

The appendices contain selections of the analyses and calculation which have been carried out in investigating the frame behaviour, both during and after the fire. The full analyses and calculations will form part of the basis of carrying out the report's recommendations with regard to "Frame Action and Structural Behaviour", "Thermal Modelling" and "Structural Modelling". Any company(s) or person(s) wishing to participate in future research activities will be welcome. Contact should be made with the Steel Construction Institute, Ascot.

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1. FOREWORD

The Steel Construction Industry Forum (SCIF) was set up in 1990 to provide a co-ordinating function between the activities of British Steel, the British Constructional Steelwork Association (BCSA) and the Steel Construction Institute (SCI) in the development and promotion of structural steel to the construction industry.

In continuation of the activities carried out by the Steel Construction Review, SCIF has identified several issues where further improvements could be made in the efficient use of steel in structures. In particular:

standardised end connections and interfaces;
the role of construction (erection) in achieving higher productivity;
the use of fire engineering in the design and execution of steel structures;
the definition and application of best practise.

The fire engineering group has representatives from various organisations involved in research and development activities associated with fire and steel structures including the following:

Mr. E.V. Girardier	Steel Construction Industry Forum
Mr. P. H. Allen	British Constructional Steelwork Association
Mr. B. A. Brown	Conder Group Technical
Dr. G.M.E. Cooke	Fire Research Station, Building Research Establishment
Mr. R.W. Gordon	Bovis Construction Ltd.
Mr. J. Hopkinson	Ove Arup & Partners
Mr. R. B. Johnson	Skidmore, Owings & Merrill Inc.
Dr. E.W. Marchant	University of Edinburgh
Mr. G. M. Newman	Steel Construction Institute
Mr. D. J. Proe	Fire Safety Engineering Consultants Ltd.
Dr. P. Robery	Stanger Consultants Ltd.
Mr. J. T. Robinson	British Steel General Steels
Mr. A. Smith	Stanhope Developments PLC
Dr. C. I. Smith	Fire Safety Engineering Consultants Ltd.
Mr. A. D. Weller	Building Research Establishment

assisted by:

Mr. P. Brett	Peter Brett Associates
Mr. J. I. Castle	London Fire & Civil Defence Authority
Mr. T. Hamilton	Richard Lees
Dr. B. R. Kirby	British Steel Technical
Dr. R. M. Lawson	Steel Construction Institute
Mr. P. Lewis	Stanhope Properties PLC
Mr. J. Oldman	London Fire & Civil Defence Authority
Mr. P. Rogers	Stanhope Properties PLC

The fire engineering group has recognised several important activities in order to further promote the use of structural steel in buildings including the following:

the potential modifications to the guidance in England and Wales Building Regulations Approved Document B

the revision of the ASFPCM/SCI publication "Fire Protection for Structural Steel in Buildings" to provide data on thickness for varying failure temperatures

production of simplified design codes

wide implementation of BS 5950 : Part 8 through promotional and educational seminars

The Broadgate Phase 8 fire is one of the first major fires to have occurred in a modern steel framed fast track building which incorporated composite steel deck/concrete floor construction. The fire has been described as severe in nature and had a duration in excess of 4 1/2 hours. The building framework retained its stability throughout the fire.

A meeting of the fire engineering group held in November, 1990, concluded that it would be worthwhile preparing a detailed report on this fire and its effects on the structure. The study would summarise the investigations carried out by:

Bovis Construction Ltd.
British Steel General Steels
Fire Research Station
London Fire and Civil Defence Authority
Ove Arup and Partners
Rosehaugh Stanhope Developments
Skidmore, Owings & Merrill
Stanger Consultants Ltd.

The group then decided to develop these reports to establish the form of the fire and the consequent behaviour of the structure. The group would like to express their appreciation of the manner in which all of these organisations readily made available their previous work and reports.

The report has been prepared by Fire Safety Engineering Consultants Ltd. on behalf of the SCIF Fire Engineering Group. The structural analysis was carried out by David Proe who was on sabbatical from BHP Melbourne Research Laboratories, Australia, during the winter of 1990/91.

The study has been funded by contributions received from:

B.C.S.A. Ltd.
Bovis Construction Ltd.
British Steel
Cryotherm Insulation
Department of the Environment
Octavius Atkinson & Sons Ltd.
Richard Lees
Stanger Consultants Ltd.
Stanhope Developments
Steel Construction Institute

Their assistance is gratefully acknowledged.

2. SUMMARY

1. The fire at Broadgate Phase 8 has provided one of the first opportunities to examine the influence of fire on the structural behaviour of a modern "fast-track" steel framed building structure incorporating composite construction. Examination of the damage which has occurred has provided an opportunity to consider the validity of design codes which have been developed from small scale fire test data.

2. The fire involved a large pre-fabricated sub-contractor's hut structure, of approximate size 40m x 12m, and caused in excess of £1.5M of structural damage, however, the total damage caused by the fire has been estimated to be in excess of £25M. The fire is known to have burnt for a period in excess of 4 1/2 hours, was "severe" for a period of 2 hours, and was fought by 20 fire brigade units.

3. The fire was unusual in character in that it occurred in a hut inside the building envelope. The sheet steel walls of the hut prevented direct access to its burning contents with water hoses, while the restricted ventilation with effectively one fire compartment inside another produced a very smokey fire which also made fire fighting difficult. It has been estimated that the flames emerging from the window of the hut had temperatures in excess of 1000^{O}C.

4. As the building was under construction, the structural fire protection was incomplete, the sprinklers were not active and the fire detection and alarm system was not operational. The building framework was lightly loaded at the time of the fire.

5. During the course of fire fighting activities a large proportion of structural fire protection was removed from the structure as a consequence of impingement of water under pressure from fire fighting hoses. The columns had not yet been clad with fire protection materials at the time of the fire.

6. Structural damage caused by the fire included distortion of a number of trusses and universal beams and axial shortening of five columns. The deflection of the trusses and beams produced dishing of the floor by up to 600mm relative to the columns, while the columns shortened by about 100mm, reducing the floor datum level of floors above the fire. The concrete floor slab separated from its metal decking in some areas but generally followed the level of its deflected supporting members.

7. Detailed studies were conducted to establish the effects of fire on material properties. It has been concluded that the materials (apart from the concrete to the first floor) showed no major loss of strength as a consequence of the fire. Detailed metallurgical investigations have been carried out to assess the temperatures reached by the quenched and tempered bolts recovered from several of the column beam connections in the areas of the fire which showed most significant damage. These have indicated that the maximum temperatures achieved by the bolts either during manufacture or as a consequence of the fire, were limited to 540^{O}C. Metallographic evidence from a sample taken from one truss has indicated that this member may have been heated to about 600^{O}C. Other bolts and steelwork would have been exposed to lower temperatures. Consequently it was concluded that the maximum temperature of the structural members was around 600^{O}C.

8. Following the principles of BS5950 : Part 8 and considering the low loads on the structure during construction, the limiting steel temperatures at which these members would transfer load to cooler parts of the structure were expected to be around 700-800°C. As the metallurgical investigation suggested that such temperatures had not been achieved, alternative mechanisms were sought to explain the structural deformation which had occurred.

9. The behaviour of the structure was satisfactory, but some of the elements had clearly transferred load to other parts of the framework, and it has been concluded that restraint within the building framework had a major influence on the structural performance of both columns and beams.

10. The columns which showed the greatest deformations were adjacent to much heavier columns. Columns heated to a higher temperature than their neighbours will attract additional vertical force and this force will be greater for greater stiffness of the beam-to-column connections. The heavy columns would generally be expected to have increased in temperature more slowly during the early part of the fire, so load would have been transferred from the heavy columns to the lighter ones. A moment frame at Levels 8 and 9 may have had sufficient stiffness to account for the load transfer which occurred.

11. Deflections in the beams are thought to have been due to restraint imposed by the action of the concrete slab as part of the composite construction. Analysis of the behaviour of the trusses indicated that there was little opportunity for these members to expand freely and consequently additional forces were generated, leading to increased deformations.

12. The assumptions used for design can be quite different from that which happens in practice. In particular, the presence of axial restraint on some members cannot be avoided for elements connected to form a framework where one part of that framework rises in temperature at a different rate to its neighbours. Regardless of the structural material used, the thermally induced forces and moments may have a substantial effect. This aspect is neglected in standard fire-resistance testing which is used as the basis for nearly all building approvals.

It should also be recognised that restraint may have a beneficial effect in the later stages of deformation of a frame, in that the same elements restraining a deflecting member will also support it and transfer load to cooler parts of the structure.

An alternative design philosophy which included restraint effects would logically need to consider the stability of the framework as a whole. The relevant design criterion would then not be loss of load capacity in individual members but the maintenance of reliable load paths through the structure. As an example, a fire-resistant steel structure could be built with internal beams and all internal columns entirely unprotected, provided perimeter beams and columns were fire protected and had adequate strength, and provided a substantial load carrying framework such as a deep truss was included at the top of the building. This truss would not need fire protection since a fire at the top level would not add load to this member and a fire in a lower level would not heat this member, assuming of course that the building was adequately compartmented. The purpose of the truss would be to carry the load shed from heated internal columns on any floor below.

13. Greater understanding and knowledge of structural frame performance can only be gained by detailed analysis. Large scale experiments would be of interest to provide information for the calibration of models. However, such experiments could only consider a small set of variables. Consequently it would be worthwhile in the future to investigate the effects of major fires in significant structures to gain a better understanding of the most important mechanisms in practice.

3. CONCLUSIONS

The following conclusions have been drawn by the group as a consequence of their studies of the Broadgate Phase 8 fire of June 23rd, 1990.

1. During the fire the building behaved as a framework of elements forming a total entity as opposed to a collection of isolated single elements. This lends support to alternative methods for the design of structures to withstand the effects of major fires to that currently adopted in Building Regulations.

2. The analysis of the structural behaviour of building frameworks following major fires will lead to a better understanding of factors which influence performance and there is a need for greater emphasis to be given to this aspect of research. Redistribution of load to less severely heated parts of the building also leads to improved performance.

3. The behaviour of connections in fire is important to ensure the continuity of the framework during the heating phase. Furthermore, the ductility of connections in the cooling phase is important to ensure the overall integrity of the structure.

4. Debonding of metal deck from composite slabs may occur in fires (or following fires) but this does not influence the anticipated fire resistance of the construction.

5. The analysis of the structural behaviour of the Broadgate Phase 8 building during the fire lends support to approaches to structural fire design as outlined in the new Code of Practice, and also the draft of Approved Document B to the Building Regulations which was circulated for public comment in the Spring of 1990.

6. In this fire the major part of the cost of refurbishment was associated with damage caused by smoke while structural damage by heat constituted a relatively small part of the total reinstatement cost.

7. The fire occurred in a nearly completed building before the permanent precaution systems had been installed. There is significant risk to both loss of life and financial loss as a consequence of fire in buildings during construction and greater emphasis should be given to temporary measures to meet this high risk.

   During the course of the study it became apparent that many more fires are occurring in temporary accommodation on building sites than is generally recognised.

8. The fire started in temporary site accommodation which did not have any detection or alarm system; prevented effective fire fighting; and contributed to the smoke which restricted fire brigade access and caused damage to finishes.

9. This study of the outcome of the Broadgate fire and the process of refurbishment has led to several recommendations for future work dealing with modelling of behaviour, cleaning of fire and smoke damaged materials, fire precautions in the construction phase and the design of temporary office accommodation.

Current codes, fire tests and Building Regulations are based upon the behaviour of single components in structures totally engulfed in fire. This study shows that when a fire affects only part of a structure and when the framework acts as a total entity, which is typical of multi-storey buildings, structural stability is significantly improved.

4. RECOMMENDATIONS

1. **Examination of Structure**
A detailed guidance document should be prepared describing the procedures, observations and tests to be carried out immediately following a fire to provide background information to help investigate the behaviour of the structure. A "flying squad" is needed to put this into effect for major fires.

2. **Frame Action and Structural Behaviour**
More research and analysis is required to study the characteristics of frame action in fires. Further understanding is required to develop and adopt the benefits of frame action relative to isolated elements.

3. **Thermal Modelling**
Analytical modelling methods should be developed and more widely adopted to study the temperature development through structural components in fires to provide input to models which analyse structural behaviour.

4. **Structural Modelling**
The use of structural models analysing frame behaviour should be extended to include analysis of fire performance of total building frameworks.

5. **Connections**
The current U.K. and European research work on connections requires further development to ensure that the necessary balance of elasticity and rigidity are maintained within the frame during the fire.

6. **Steel Deck**
Further study and investigation should be carried out into the effect and the repairs necessary in a fire damaged composite floor.

7. **Cleaning**
A guide dealing with the effects of acidic sooty gases, corrosion and cleaning of various building materials would be of assistance to those involved in reinstatement of buildings following fires.

8. **Fire Precautions Aspects**
Measures to improve general fire safety and reduce financial loss are required for building sites. (See Addendum).

9. **Temporary Accommodation**
All forms of construction for temporary accommodation inside buildings should adopt low combustibility materials which only release small quantities of smoke when exposed to fire. Guidance should be given on the forms of construction which are acceptable for use in different situations.

10. **Appraisal Techniques**
Broader adoption of fire engineering appraisal techniques for specifying fire resistance periods to satisfy Building Regulation requirements is justified.

5. INTRODUCTION

During the last decade there has been a most significant growth in the use of steelwork in multi-storey commercial buildings in the U.K. The requirements to produce commercially attractive buildings necessitated the introduction of methods of construction widely adopted in North America. The main commercial requirements to be satisfied were associated with cost, speed of construction, and long span column free spaces were solved by the use of composite methods of construction involving metal deck/lightweight concrete floors supported from beams. Composite action is achieved by the use of shear studs welded to the top flange of the beam. The improvements in the method of construction were supported by research, testing, and design codes; in particular, aspects of behaviour in fire. Such buildings have been designed using the principles outlined in BS 5950 : Part 1 (1) and Part 3 (2). Very recently BS 5950 : Part 8 : 1990 (3) has been published to provide specific design advice on the behaviour of steel framed buildings in fire conditions. The guidance provided in the above standard is based upon standard fire resistance tests performed in relatively small specimens following the BS 476 : Part 20/21 procedures.

During the morning of Saturday, June 23rd, 1990, a major fire occurred in the Broadgate Phase 8 building which forms part of the prestigious new Rosehaugh Stanhope office development built around the Liverpool Fire Station in London. This building had been designed by Skidmore Owings and Merill using a composite steel/concrete framework, following guidance in the above Codes.

The fire occurred during the final stages of the construction programme. The building framework was complete, and curtain walling had been installed around most of the building. Most of the fire protection works to the structural framework of the building had been completed; but some columns in the neighbourhood of the hut were unprotected. The automatic fire detection and alarm system, and the sprinkler protection system was not yet operational when the fire occurred. Extensive work had also been completed in fitting out the building shell with services and finishes in readiness for handover to the building users for final fit-out of the office facilities. The fire occurred in the sub-contractor's office facility located at Level 1 in the building, and burned severely for a period in excess of 2 1/2 hours. Twenty fire brigade units attended the fire and the time between their attendance and the stop notice was 5 1/2 hours.

Following the fire, limited damage had occurred to the structural framework of the building, including small deformations to the heads of columns, larger deflections to floors and truss assemblies, and small deflections to many secondary beams.

Detailed studies have been made of the fire and its effects on the structure and building materials.

The fire engineering group of the Steel Construction Industry Forum decided at the meeting held on November 2nd, 1990, that the studies carried out by various consultants should be summarised into one consolidated report which could be used by others in their studies of fire and steel framed buildings.

The principal motives for the study were as follows:

1. It was probably the first major fire which has occurred in a modern fast track building structure in the U.K. which incorporated composite steel deck/concrete floor construction.

2. During the design of the structural fire protection system detailed analyses of the structural behaviour of trusses and beams had been carried out by the Steel Construction Institute in conjunction with Fire Safety Engineering Consultants Ltd. to define failure temperatures at the anticipated floor loading under fire conditions, with voids between deck and top flange unfilled. Hence the anticipated fire performance of the structure had been studied in depth in advance of the fire.

3. The fire protection thicknesses for the beams and trusses were specified to satisfy one hour fire resistance, as part of an approach which used the contribution of the sprinkler system to satisfy the functional requirements of Regulations.

4. Draft BS 5950 : Part 8 : 1990 "Structural Use of Steelwork in Building : Part 8 : Code of Practice for Fire Resistant Design", had been prepared using the results of both small scale test results on material properties, and fire resistance test data on small assemblies to produce a design method for the design of modern steel framed buildings. Little opportunity had existed to check the applicability and accuracy of the Code in full scale building assemblies involved in fires.

5. Several articles have appeared in the popular press describing the fire, and it was appropriate that a definitive statement be issued regarding the extent, cause and type of damage which occurred.

This report provides a summary of the data collected and conclusions drawn by many of the organisations including:

the Fire Brigade observations and activities during the fire;

the conclusions of the Fire Brigade Investigation Team;

the characteristics of the fire as studied by Ove Arup and Partners (4);

the analysis of structural behaviour made by S.O.M. (5) and FiSEC;

the structural material characteristics of materials used in the building framework as established by Stanger Consultants Ltd. (6);

the detailed metallurgical studies carried out on steel components by British Steel Technical (7).

Since this fire, various other major fires have occurred in buildings nearing completion during the construction phase. This has caused the authorities and developers to consider Fire Precaution aspects during the Construction Phase in more detail to ensure that more emphasis is given to fire problems. Although these aspects were beyond the scope of the terms of reference for this study the group believed that this aspect should be mentioned in their report. Consequently an addendum has been prepared highlighting some key points which should be considered to improve current standards.

6. BUILDING DESIGN

6.1 Building Description

Phase 8 of the Broadgate development involved a multi-storey office block built adjacent to Phases 7 and 14 of the project. The building is built over the main British Rail railway line which provides services predominantly from the East of London. Phase 8 is built facing Bishopsgate on the East and Exchange Square on the West. The building is separated on the South from Phase 7 by a fire resisting wall, and is spaced away on the North from Phase 14 at levels above ground level. A sketch showing the location of the building with respect to other phases of the development is shown in Figure 1. Figure 2 shows a general building plan showing the location of the hut on Level 1.

Plate 1 shows a photograph of the building from Exchange Square. The building has 14 main levels, along with some internal mezzanines. There is a raft which spans across the railway tracks. This level is used for plant and an access road from Primrose Street through Phase 14 to Phases 6 and 7 for retail purposes. There are 13 floors above the ground floor.

Sketches showing sections through the building are shown in Figure 3.

The building has an overall size of 84 x 55m with a height from the raft to the parapet of the roof of 65m. Figure 4 shows a structural plan at Level 2, i.e. the floor affected by fire attack.

The building has a structural steel framework. The universal columns support fabricated trusses and universal beams, which in turn support a steel deck for the light weight concrete floors. The use of trusses were preferred in this building to provide a span which was light, long and relatively column free.

The openings in the 1000mm deep trusses provide access for services, and thereby generally allow a floor to floor height of 4100mm. Floor to floor heights of 5m are used at first to second, second to third, and third to fourth floors.

The building is braced in the East/West direction by triangulated steel members in the centre core and in the North/South direction by the moment connections between beams and columns on the East and West sides.

The floor system consists of a composite slab made up of Richard Lees Ribdeck 60 trapezoidal deck which spans a maximum distance of 3000mm between beams. The deck is reinforced with A142 mesh located to give 30mm cover from the top of the lightweight concrete slab, density 1750 kg/m^3, which has a minimum thickness over the ribs of 70mm and an overall thickness of 130mm. The slab was designed following the guidance presented in the SCI document "Fire Resistance of Composite Floors with Steel Decking" (8), to provide 1 1/2 hour fire resistance.

The building is clad with a prefabricated curtain walling system fixed from brackets attached to the floor slab. The main shafts are constructed in fire resisting plasterboard constructions. None of the fire doors to the staircases or lifts were in position when the fire occurred.

The steel structure supporting the floor was made up of a mixture of lattice trusses and universal beams running east west and universal beams running north south on the fire floor. The columns were spaced on a 13.5 x 6m grid. The 1m deep trusses had top and bottom chords of Tee sections with double angles as bracing members (see Figure 5).

The column members (Grade 50B steel) utilised on Level 1 in the area of the building which was involved in fire had serial sizes as follows:

C32, C33, C34	356 x 406mm x 287 kg/m
D32, D33, D34	356 x 406mm x 634 kg/m
E32, E34	356 x 406m x 235 kg/m
E33	356 x 368 x 177 kg/m

The heavier columns used on Gridline D were a continuation of the heavy transfer structure used to span across the railway tracks. The splice to lighter columns was made between Levels 2 and 3.

The building design adopted fire engineering design principles, which included recognition of the benefit of the sprinkler system could have in controlling an outbreak of fire in the early stages. The vertical structure (columns, staircases and vertical shafts), was designed to satisfy 1 1/2 hours fire resistance. The floor slab had a thickness associated with 1 1/2 hour fire resistance. The steel beams and trusses were to be protected with a lightweight fire protection product with a thickness specified to satisfy 1 hour fire resistance. The voids formed between the steel deck and the top flange of the beam were not filled with insulating material. These reductions in specification for the beam and truss structure was based upon fire severity calculations which assumed a fire load density of 700 MJ/m^2 for the office (an 80% fractile value) along with appraisal of failure temperatures using BS 5950 : Part 8 concepts. The failure temperatures calculated took into account the higher temperatures at the top flange which would be partially unprotected.

The design approach therefore recognised both the importance of the vertical structure in controlling the overall stability of the building framework and the benefit of the sprinkler system in controlling fire severity, while realising economies in the cost of the fire protection of beams and trusses. (All the horizontal steel structure was protected with minimum available thickness of mineral fibre slab.)

The approach was supported by analysis of statistical data on fires in offices and comparison of the probability of success of the system versus the conventional design approach based upon 1 1/2 hours fire resistance for all elements of structure without a sprinkler system as recommended in Approved Document B (9).

The building has a sprinkler system but this was not installed at the time of the fire.

The beam structure was fire protected using a mineral slab system of density 200 kg/m^3 supported using noggings and specially designed screws. The columns were to have been protected with plasterboard. It has been stated that "The entire structure except first floor columns and all perimeter beams, were fire protected at the time of the fire.

The automatic fire detection and alarm system was not installed when the fire occurred.

The building was "lightly loaded" when the fire occurred. It was unoccupied by tenants, and consequently the loads imposed by the contents of the building were lower than the design values. The positions and additional weight from an attrium floor, and a sub-contractor's hut on the floor above are noted in Figure 6. Detailed surveys of the building were conducted following the fire and Figure 7 shows the deflection in the slab; Figure 8 shows the position of cracks in the floor slab; and Figure 9 shows the positions of the bolts connecting beams and columns which were subsequently tested by British Steel.

6.2 The Site Hut

The site hut, 40 x 12m and was divided into 13 office facilities for use by sub-contractors. The hut was raised above the structural slab to allow access underneath for fixing services at the floor below.

The raised floor was built using a structural steel frame which supported timber joists and the plywood decking. The steel frame consisted of 150 x 100mm RSJ's which were raised 600mm from the floor using 150 x 100mm RSJ stilts. The steel joists were placed at 2.4m centres and these were used to support the 150 x 50mm timber joists. The walking surface was provided by 19mm thick plywood installed as sheets 2.4m long x 1.2m wide. This provided a basic surface for the construction of the hut.

The walls of the hut were prefabricated sandwich panel consisting of sheet steel laminated to a 38mm thick expanded polystyrene board of density 20 kg/m^3. The external and internal sheet steel linings were plastic coated and had a total thickness of 0.7mm, i.e. Colorcoat. The polystyrene board was of "A" quality as defined in BS 3837 : Part 1 : 1986 (10), where a performance requirement is that in BS 4735 (11) test specimens 150mm x 50mm x 13mm subjected to a small flame should show an "extent burnt" of less than 125mm. This requirement is met by the incorporation of the flame retardant additive or other appropriate modification. In this test the sample is supported horizontally on a wire mesh and exposed to a flame 38mm high from a wing tip burner sited 12mm below one end of the sample. The extent and rate of flame spread along the long axis is noted. Hence this test is a small scale reaction to fire test.

The wall panels had an overall size 2.42m high x 1.21m wide and were linked together using profiled steel sections. Standard panels are available and include window and door openings. The panels are screwed to the frame members and the roof construction is made up using a similar method using the same panels supported from cold-formed joists.

A sketch showing the basic construction of the hut is shown in Figure 10 while Figure 2 shows an architectural plan at Level 1 showing the approximate layout and sub-division.

Plate 2 shows a photograph of a similar hut on the floor above after the fire. This hut suffered slight damage as a consequence of fire destroying the windows on both first and second levels.

7. THE FIRE INCIDENT

7.1 Time Line

The following information lists the main sequence of events from the records kept by the Fire Brigade.

The fire occurred during the night of 22nd-23rd June, 1990. The weather was reported to be fine with no significant wind.

00.15	Time of discovery. The precise cause of the fire is unknown. It is thought that possible ignition sources were: (a) electrical fault, (b) overheating of clothes in drying room of office, (c) careless disposal of smoking material(s).

The LFCDA's Fire Investigation Team discounted (a) and (b) as likely causes, and concluded that (c) was the most likely source of ignition.

The official Fire Brigade log was as follows:

00.33	Initial Call
00.44	"Make Pumps 4"
00.51	Informative: "Building under course of construction. 10% first floor alight".
00.59	"Make Pumps 6. Persons reported. Cylinders involved".
01.09	Informative: "Approximately 6 propane cylinders and 2 oxyacetylene cutting sets believed involved.
01.21	"Make Pumps 10".
01.35	"Make Pumps 20".
01.41	Informative: "Upper floors smoke logged. BA crews searching".
02.26	Informative: "7 jets and hydraulic platform monitor in use".
02.54	Informative: "Steady progress being made. Deep seated fire remains in 1st and 2nd floors. Remainder of building smoke logged. 7 jets and breathing apparatus in use".
03.44	"Fire surrounded".
04.24	Informative: "Steady progress being maintained. Deep seated fire remains on 1st floor. Part of 2nd floor in danger of collapse. Crews have been withdrawn from 1st and 2nd floors".
06.01	Stop message.

Initial fire fighting operation took place inside the building, but crews were forced to withdraw when visibility was severely reduced by smoke. Problems were encountered in locating fully operational dry risers as a consequence of unfamiliarity with the building layout and the access route, hoardings and inadequate signs. The main initial fire fighting took place from outside the building using a hydraulic platform located in Bishopsgate.

Statements taken from fire-fighters indicate that the windows on this elevation of the building were still intact at the time of the arrival of the first appliances. A decision was taken to break these windows in order to allow use of a water monitor from the head of a Hydraulic Platform. The windows to the Bishopsgate elevation were broken at approximately 1.20 a.m.

Fire fighting activities were pursued from the Exchange Square elevation at a later stage around 1.45 a.m.

A series of large bangs were heard during the course of the fire. The most significant bang occurred at around 1.50 a.m. and this was associated with some aspect of structural movement, probably column deformation, cracking of the floor slab, or slipping of bolted connections.

The Fire Brigade concluded that most of the structural damage had occurred by 2.00 a.m.

Plate 3 shows a photograph of fire fighting activities from the Broadgate elevation.

7.2 The Progress Of The Fire

The progress of the fire through the relocatable buildings is largely unknown. The fire probably burned "severely" for a period of around 2 1/2 hours, i.e. from approximately 1.00 to 3.30 a.m. when the "fire surrounded" notice was issued. Following the fire most of the combustible material had been consumed, although parts of the timber floor were intact.

The fire duration coupled with the complete burning of combustible material suggests that fire spread along the unit was relatively slow. Under free-burning conditions, the contents of an office would be totally consumed in a period of less than 1 hour. Studies of real fires such as the Interstate Bank in Los Angeles (12), along with results of tests involving either timber, or materials and furniture, used in offices generally involve a localised total burn-out in a period of less than one hour (13).

The hut construction involved separating the building into sub-offices using plasterboard walls. Consequently fire spread could have occurred by:

a) spread around openings for services, or wall/floor or wall/roof junctions;

b) spread via either the roof, wall or floor construction.

The roof walls of the units were made of polymeric sheet steel sandwich panels, and under fire conditions combustible gases would be released which could easily be ignited.

The floors of the units were timber raised from the concrete floor to provide a void for access. Hence, fire spread could have occurred directly via the floor.

The main windows of the units faced the wall separating Buildings 7 and 8 and the central staircase core, and hence most were inaccessible for fire fighting operations from Bishopsgate and Exchange Square. Consequently the fire duration was long and most combustible material was consumed.

The walls separating the various parts of the hut were constructed using a plasterboard stud wall construction. In the event of a major fire the walls would have been expected to collapse, as they would have a nominal 1/2 hour fire resistance. Hence fire could have spread in a progressive manner.

The main features of the fire damage to the huts following the fire were as follows:

1. The panel and roof components which were assembled together to form the hut had little integrity. The structure of the hut was destroyed at both ends, the end facing Exchange Square showing the greater damage.

2. The timber floor showed the most severe damage at the Exchange Square end, and progressively lesser damage towards the Bishopsgate end. In some areas the timber joists and boarding were only lightly scorched by the effects of the fire, but in others the floor was totally destroyed.

The extent of damage to the hut structure reflects:

a) the initial fire fighting activities from Bishopsgate; and

b) the effects of the water hoses on the lightweight panel walls and roof as a consequence of the fire fighting activities from outside the building.

7.3 Fire Load Density

The fire load density in the site hut cannot be defined with any precision. The units were used as offices where a mean fire load density of 20 kg of wood/m^2 can be adopted. As the offices were small, it is likely that a higher value should be adopted, say 25 kg/m^2.

The floor used in the building consisted of timber joists and the 19mm thick floor. This would contribute around 15 kg/m^2 to the overall fire load density.

The polymeric foam used as thermal insulation in the composite panels forming the walls and roof of the site hut had a thickness of 38mm and density of 20 kg/m^3. The material has heat output of 40 MJ/kg, i.e. a factor doubly higher than timber. Timber has a heat output of 18 MJ/kg.

Hence the approximate fire load density has been calculated as follows:

Office Contents 40 x 12 x 25 = 12000 kg

Office Floor 40 x 12 x 15 = 7200 kg

Total = 19200 kg

Calorific Value = 345,600 MJ

Wall Area = 104 x 2.4 = 250m^2

Roof Area = 40 x 12 = 480m^2

Total Area = 730m^2

Weight of polymeric foam = 730 x 0.038 x 20 = 555 kg

Calorific Value = 22,200 MJ

Total Calorific Value = 367,800 MJ

Fire Load Density = 367,800/480 = 766 MJ/m^2.

7.4 Fire Severity

In order to obtain an estimate of the likely temperatures achieved by the fire, calculations have been performed using the procedures described in the S.C.I. publication "Fire and Steel Construction - Fire Safety of Bare External Structural Steel" (14). This document provides a method of calculating the temperatures of the flames which project from windows of burning compartments with known fire load densities, floor areas, bounding surface areas, and openings. The site hut was divided into sub-compartments of varying dimensions with varying sizes of windows. In order to check the sensitivity of fire temperature to both compartment configuration, fire load and size of openings, calculations have been performed adopting a computer programme for simplicity to encompass the range of typical variables expected. Three typical compartments (see Figure 11) were defined as follows:

Case A

A compartment of width 7.2m, depth 11m, and height 2.4m, incorporating two 1.0m high x 0.9m wide windows and a 0.75m x 2m high door.

Case B

A compartment of width 5.4m, depth 11m, and height 2.4m, incorporating a single 1.0m high x 0.9m high window and a 0.75m x 2m high door.

Case C

Involving a 3.6m wide, 5.4m deep, 2.4m high, compartment with a single 0.9m x 1.0m window and a 0.75 x 2m door.

Case D

A compartment of identical size to A incorporting two 1.0m high x 0.9m wide windows and a 0.75m x 2m high door on each of the 7.2m wide elevations.

Case E

A compartment of identical size to B incorporating single 1.0m high x 0.9m wide windows on each of the 5.4m wide elevations with a single 0.75m x 2m high door.

Calculations of the temperature at the window opening have been performed for fire load densities of 20, 30, and 40 kg/m^2, i.e. 360, 540 and 720 MJ/m^2. Table 1 presents the calculated fire temperatures both within the compartment and at the window.

These calculations show that the fire was "ventilation controlled". The fire temperatures at the window are higher than those calculated in the room indicating that combustion gases are igniting at the window as they leave the compartment. As the fire is ventilation controlled increasing the fire load density does not increase the temperature of the fire at the window.

The calculated fire temperatures at the window are based upon the combustion of gases in open air with a ready supply of oxygen. The Broadgate Phase 8 compartment was enclosed to some extent, as the glazing was complete, around both the external elevation and the lower atrium. Openings were limited to doors to staircases and some gaps at the curtain wall floor slab junction. Hence, even at the window of the hut combustion may well have been ventilation controlled giving lower temperatures, and a very smoky environment. Ventilation would have been increased by breaking the windows during fire fighting.

Flames emerging from the windows of the hut are likely to have had temperatures in excess of 1000°C, however fire temperatures inside the hut are likely to have been considerably lower.

While the fire load density calculated is similar to that anticipated for an office fire, the fire characteristics were different from those which would have been expected in the completed building.

The construction of the hut which involved a sandwich panel sheet steel construction restricted fire-fighting activities whereby the water jets could not be directed to impinge on the burning objects. Consequently the fire duration was longer than might have been anticipated. The combustion of polymeric foams and coatings in a ventilation controlled fire created a very smoky environment.

In the completed building any fire should be controlled in its early stages by the sprinkler system. Should the sprinkler system malfunction fire growth would be rapid and flash over could eventually occur causing the fracture of many of the windows around the perimeter of the building. However, before this stage, the efforts of fire fighting would be more successful as the water applied would not only cool the fire but would also pre-wet combustible items not already involved.

In this incident the fire fighting efforts were not able to be totally effective in controlling the fire, but rather have restricted the temperature rise of the main structural members of the building.

Several fire tests have been conducted to establish the fire temperatures achieved in office fires, but a limited number of tests have been performed on prefabricated buildings. The Fire Research Station (15) have completed a series of tests on caravans where temperatures in excess of 1000°C were realised in tests performed on two types of mobile home around 10 minutes after ignition in "well ventilated" fire tests.

8. FIRE DAMAGE

8.1 Structure

The structural damage was restricted to the main area of the fire at the south end of Broadgate Phase 8, adjacent to Phase 7, on the floor on which the fire occurred between Levels 1 and 2. The damage necessitated the replacement of columns, beams, trusses and floor slabs over an area of the building 40m by 20m.

The facade of the building was largely intact following the fire. Only four windows had been broken, either by the fire or during fire fighting activities.

Significant burning or charring of timber had occurred around the timber frames supporting atrium glazing.

Photographs showing the damage are shown in Plates 4 - 12.

Hot gases from the fire did spread to Broadgate Phase 7 at the west end of the structure, to an extent which activated the sprinkler system. These gases apparently passed through an opening at the top of the lightweight wall block. It is unknown whether this gap had been fire sealed at the time of the fire.

The extent of replacement of the structural elements after the fire was as follows:

Beams and Trusses	44 number (with a further 7 strengthened)
Columns	5 number replaced
Deck Floor Slab	1520m^2

The damage to beams included overall bowing which was fairly even along the length, sagging over only part of the length, local buckling at the ends along the bottom flange and in some cases web lateral distortion. There was no clear evidence of the formation of midspan plastic hinges.

The trusses showed local buckling of the compression diagonals leading to sagging of part of length of the truss and in several cases curling up of the end of the truss after buckling of the adjacent compression diagonal.

The five damaged columns all formed large buckles within about 0.3m of the top connection. They also probably underwent considerable plastic shortening in these regions, as the reduction in their length was of the order of 100mm..

The floor slab suffered debonding of the steel deck in all areas of the fire zone. The deformation of the concrete generally followed that of the supporting members below with further deflection of the slab between these supporting members (see Table 2). This deflection resulted in horizontal and vertical cracking within the concrete. The vertical cracks tapered to zero width at the top face. Three large cracks were observed in the floor slab from the top surface of Level Two (see Figure 8). These large cracks resulted in stepped openings where the reinforcement had fractured as a consequence of a ductile failure.

A detailed photographic survey of damage to connections has been done by SOM and this shows many connections which have undergone large rotations in the area of the building where the structure was replaced.

In areas where the first floor concrete had been overheated it was cut out to a depth of 20mm and relaid.

8.2 Fire Protection Material

The material which was used for the fire protection of the trusses and beams was a mineral fibre slab fixed in place using spiral screws. The mineral fibre slab had a density of 200 kg/m^3 and generally the thickness of product utilised was 20mm. During the course of the fire and the fire fighting activity a large proportion of this product apparently became detached from the structure. Photographs of the beams taken immediately after the fire indicate significant areas of detachment (see Plates 4 - 12). The product has been subjected to rigorous fire test evaluation using the BS 476 : Part 20/21 procedures, (16, 17) following a general testing programme involving loaded sections and unloaded exploratory members outlined in the ASFPCM and SCI publication "Fire Protection for Structural Steel in Building (Second Edition) (18).

These tests included a loaded beam test on a 305 x 165mm x 42 kg/m beam protected with 20mm of board. The assembly showed a fire resistance of 100 minutes when tested at the maximum permissible design stress. Heating of the member continued for a period of 180 minutes. At the time when the load was removed from the beam its deflection was 110mm, and the rate of deflection was 14mm per minute. The test was carried out at the Warrington Fire Research Centre where the beam span was 4.5m, so consequently this deflection of 110mm relates to span/41. When the load was removed the fire protection system was intact, except for a 50mm wide opening, but after 112 minutes a large proportion of the fire protection became detached from the beam. This characteristic is sometimes typical of that expected for a boarded system of fire protection supported from noggings.

The extent of the detachment of the product from the structure at Broadgate could well have been anticipated as a consequence of a number of factors:

1. The mineral fibre boards are generally bonded together using resin which has a tendency to degrade at elevated temperatures.

2. The mineral fibres in the boards are accepted to have sintering temperatures around 700^{o}C and above this temperature the products lose their fibrous texture and become powdery in nature.

3. The use of numerous high pressure water hoses in fire fighting activity would have caused the fire protection material to become saturated and consequently its strength would have been reduced. Subsequent impact from high pressure hoses has undoubtedly caused the material to become detached from the structure. This conclusion is confirmed by examination of the photographs which show the extent of the intact fire protection after the end of the fire. The use of the windows as the main openings for the fire fighting activities resulted in shielding of certain parts of the structure from the effects of the hoses. Consequently the extent of removal of fire protection from the lower flange of the trusses is much greater than the removal of fire protection from the upper boom. The structural fire protection has been detached from the structure in areas of the building which would not have been subjected to severe fire attack, or significant deflection of the structure, and therefore the action of the hoses is the only explanation which can be offered for the detachment of the material.

4. The fire had an unusual character in that the burning materials were contained inside the hut with the sheet steel external skin, so consequently the application of water from the exterior elevations of the building did not extinguish the fire. It is likely that the fire continued for a long time after the removal of the fire protection by the action of the hoses.

5. Fire tests have been conducted on steel members protected with the fire protection product which have been exposed externally for a period of one year without any detrimental effect on performance. Furthermore, the product has been saturated by water in another building affected by fire without detachment.

8.3 Smoke Damage

Significant smoke damage occurred throughout the entire building. The staircase and lift shaft openings allowed smoke to rise to Level Two and then smoke circulated throughout the atrium to the rest of the building. This caused large-scale damage to finishes. The smoke deposits were of an acidic nature containing a high concentration of chloride.

8.3.1 Smoke Output

The amount of smoke emitted from flaming materials can be quantified by a parameter referred to as the smoke conversion factor (19):

For wood (Douglas Fir) $e = < 0.01 - 0.025$

For Polystyrene $e = 0.16$

Mass of smoke $= e \times$ Mass of fuel consumed

The mass of timber was estimated as 19,200 kg.

The mass of foam was estimated as 550 kg.

Hence smoke from timber $= 19200 \times .02 = 384$ kg

Smoke from polystyrene $= 550 \times .16 = 88$ kg

Hence larger amounts of smoke would have been produced by wood than the polystyrene foam. A high value of smoke conversion factor for wood of 0.02 was chosen from the range to reflect the ventilation controlled combustion conditions.

The potential sources of the acidic soot containing chloride deposits (see 8.3.2) would therefore have been electrical cables, the pvc coating on the steel sheets and other components, possibly including the floor tiles.

8.3.2 Corrosion Effects Of The Sooty Deposits

The possible effects of acidic soot deposits on the durability of steel, galvanised items and concrete caused some concern. Test work carried out during the refurbishment exercise indicated that sooty deposits on the steel were acidic in nature having pH values as low as 3.0 and also contained chloride deposits with concentrations in excess of 3000 micrograms/cm^2.

The source of the acidic chloride deposits is likely to have been from the combustion of polyvinyl chloride materials from which hydrochloric acid is readily liberated. The theoretical hydrochloric acid content in the absence of plasticisers, extenders and fillers in the PVC, is about 50% by weight of material consumed.

The likely corrosion damage by acidic soot deposits on unprotected structural elements and galvanised steel components are as follows:

a) Structural Steelwork

The corrosion rate of iron and steel is not greatly affected by solution pH values in the range 4 to about 9.5. Over this range, steel does corrode, but the rate is not pH dependent. At the pH value measured, the general corrosion of the steel over and above that caused by water condensation under high humidity conditions would not have been affected significantly.

It is possible that some localised pitting corrosion occurred, more so as the steel was in the mill-scaled condition, but any pitting damage would not have been sustainable even allowing for auto-catalytic chloride pitting corrosion mechanisms.

Carbon (soot) is cathodic to steel and it is possible that after the fire and whilst any soot-affected steel might have remained wet, some soot-induced localised corrosion took place. This would have depended upon the period and duration of wetting or condensation prior to cleaning after the fire. Some additional light rusting could be anticipated while the steel structure was wet and the building was drying out following fire fighting and cleaning activities.

However, corrosion damage on the structural steel elements is unlikely to have been significant. The building was quickly reinstated and during its life will be environmentally maintained, the likelihood of condensation, after a period to allow for temperature and humidity stabilisation, would be minimised and, thereby, any further corrosion damage will be limited.

Chloride contaminated soot may still be present in crevices posing a potential risk of corrosion should condensation occur.

b) Galvanised Floor Decking

Zinc, because of its amphoteric nature, is not as tolerant towards solution pH values as is steel. Stable protective films are formed on zinc only within the pH range from 6 to 12 and beyond these values, zinc is attacked by both acid and alkali media.

In the case of the galvanised decking there was concern that grey or white corrosion products where acid soot deposits occurred with the probability of significantly reducing zinc coating thickness on affected areas. With the low pH values reported, some attack on the zinc was anticipated because the rate of zinc corrosion is rapid at pH values more acid than about pH = 6.

Zinc is also prone to white rust corrosion damage under damp sheltered conditions both in contact with itself and with other materials. Zinc is less affected than steel in its galvanic corrosion in contact with carbon (soot).

Cleaning was effected very early in the reinstatement exercise, and comprehensive measurements made of zinc coating thickness. No significant reduction in zinc coating thickness was found as a consequence of vapourisation, corrosion and cleaning. Where necessary supplementary corrosion protection systems were applied.

The galvanised metal deck in contact with the concrete will have been attacked by the alkaline nature of the concrete, although the level of attack will be slight and will have ceased when the concrete became dry. The corrosion rate of zinc is as great in alkaline solutions with pH values in excess of 12 as it is in acid solutions with pH values less than about 6.

c) Corrosion Damage to Concrete

Most published work on concrete corrosion is concerned with the effect of chloride on steel reinforcement.

The hydrochloric acid in soot will undoubtedly have reacted with the alkaline cement to liberate chloride ions (Ca, Na, K). However, given the quantities of acid chloride necessary to cause surface erosion or reinforcement corrosion, damage due to these effects is unlikely to be a factor for serious consideration.

Penetration of acid soot down cracks in concrete would be of more concern, as the cracks lead directly to the reinforcement. Also, heat from the fire can result in a loss of protective alkalinity. Both factors are important where the moisture content of concrete is likely to be high or may become so in the future. The concrete decking at Broadgate Phase 8 was internal and hence these factors are not of concern.

d) Summary

Based upon the pH values measured it is unlikely that the structural steel elements have suffered any permanent corrosion damage or that their future durability is at risk. The initial concern that the galvanised steel deck may very well have been corrosion damaged by acidic soot deposits to the extent of the formation of unsightly and perhaps voluminous white or grey rust deposits beneath which, the zinc coating may have lost much of its original thickness did not arise. In-situ metal coating measurements indicated no loss in zinc coating thickness.

It is unlikely that the concrete floors will have been seriously affected by acid attack at the pH levels quoted and because the source of acid was restricted to the fire event and probably due to limited amounts of burning PVC. Because concrete is within the building, corrosion of reinforcement due to the effects of loss in alkalinity or chloride penetration are unlikely once the building has been fully reinstated and all internal floors fully dried out.

The Broadgate Phase 8 building has potential life in excess of 100 years, which could be significantly reduced by detrimental effects of corrosion. Hence, it was important that all potential problems were identified, investigated and resolved in order to maintain the investment value of the structure.

9. MATERIAL TESTING AFTER FIRE

9.1 Structural Steel

Tensile specimens taken from samples of trusses damaged in the fire were tested by Stangers (20) and had strength values as follows:

	Yield Stress (N/mm^2)	Tensile Strength (N/mm^2)
Truss CD7	423	537
Truss DE6	419	550
Truss CD4	357	513

BS 4360 Grade 50B for sections up to 16mm thickness has a minimum yield stress of 355 N/mm^2 and tensile strength range of 490-640 N/mm^2. Hence all these samples were within the specification range.

Metallographic evaluation carried out by Stangers indicated no change in microstructure of the structural steel. The main changes (21, 22), which might have been expected include:

1. Spheriodisation of pearlite which would begin to occur at temperatures above 600^{o}C. [Spheroidisation could occur at lower temperatures in highly stressed members via a stress enhanced low temperature phenomenon.]

2. Mixed grain growth as a consequence of under-annealing in the temperature range 725-850^{o}C.

Detailed metallographic examination of five specimens taken from trusses have been examined by British Steel who have confirmed that four of the five samples examined exhibited no evidence of spheriodisation. However, sample C2 taken from truss CD5 did show some slight evidence of the early stages of pearlite spheroidisation. Further detailed examination of this specimen using the scanning electron microscope has confirmed the early stages of spheroidation which suggested that the member has been exposed to temperatures marginally in excess of 600^{o}C. The temperature in the member was definitely not in excess of 723^{o}C where the transformation from body-centred cubic to face-centred cubic structure occurs. The extent of spheroidisation depends on the temperature and time of exposure and was typical of that caused by exposure for a long period at temperatures around 600^{o}C, or alternatively, for a shorter period at a slightly higher temperature, say 620^{o}C.

9.2 Concrete

Stangers (23) measured compressive strengths of concrete core samples taken from the Level 2 floor slab. Of 15 samples taken from the region above the fire, 7 were already cracked and the other 8 samples showed a range of compressive strengths from 40 to 54 N/mm^2. These values may be compared with 11 samples taken from the same level further from the fire zone of which none were cracked and the compressive strength ranged from 45 to 57 N/mm^2.

Despite the natural variability of concrete properties, all cores satisfied the strength requirement of Grade C35, indicating the physical properties of the bulk of the slab was unaffected by heat. It should be noted that the large number of cracked cores from the region of the fire suggests deformation has had a greater effect on the concrete.

9.3 Bolts

Extensive metallurgical testing was carried out by Stangers (24) to establish the strength of bolts recovered from various parts of the building.

The yield and ultimate loads for the Grade 8.8 bolts were as follows (minimum and maximum values):

	Yield Loads (kN)	Ultimate Loads (kN)
M20	165-205	210-251
M24	239-329	317-387
M30	340-390	490-520

According to BS 3692 : 1967 (25), the minimum proof loads for Grade 8.8 M20, M24 and M30 bolts respectively are 140 kN, 201 kN and 321 kN. Friction grip bolts manufactured to BS 4395 : Part 1 (26) specify similar mechanical properties for Grade 8.8 bolts.

British Steel Swinden Laboratories have carried out a metallurgical evaluation in an attempt to establish the temperatures achieved by several quenched and tempered bolts recovered from the building (7). The bolts had previously been tested by Stangers. The table below shows their strength properties as determined by Stangers. Figure 9 shows the positions of the bolts within the building.

Bolt Reference No.	Diameter/Length	Grade	Yield Load (kN)	UTS (kN)
34	M20 x 80	8.8	195	231
34 Re-test	M20 x 80	8.8	175	233
36	M20 x 55	8.8	180	215
40	M20 x 110	8.8	182	217
37	M30 x 110	8.8	376	510
47	M30 x 115	HSFG	305	460

During their manufacture Grade 8.8 bolts with mechanical properties conforming with BS 3692 are generally oil-quenched from an austenitising temperature of 900^{O}C. The bolts are then tempered in order to establish the required strength and ductility at a temperature in the range 440^{O}C-600^{O}C depending on steel quality. The strength characteristics of bolts are influenced by the heat treatment temperature. Figure 12 shows a plot of strength vs. tempering temperature for Grade 8.8 bolts heat treated for 1/2 hour up to 4 hours. This figure shows that the heat treatment temperature is much more significant than the heat treatment time. The mechanical properties of quenched and tempered steels can also be characterised by their hardness in the Vickers hardness test. The hardness test involves a small scale test which is easily repeated on relatively small specimens. Hardness is related to tensile strength by a relationship presented in ASTM E140 (27).

BS 3692 (25) specifies both minimum and maximum hardness values, Hv30 of 225 and 300 respectively.

The as-received hardness values of the bolts as manufactured were as follows:

Bolt No.	Hv30
34	286
36	269
37	267
40	271

Hence, all hardness values were within relevant specification requirements.

HSFG 30mm diameter bolts are required to have a minimum Hv30 of 225 and maximum Hv30 of 292 for general grade bolts manufactured to BS 4395 : 1969 : Part 1 (26). Bolt No. 47 had an as- received hardness of 257, i.e. within specification.

The experimental procedure used by British Steel involved the use of 10mm slices of bolt shank. The hardness of the as-received specimens was checked at nine positions across the diameter of a 20mm diameter bolt and fourteen positions across the diameter of a 30mm diameter bolt. The individual bolt sections were then heat-treated at temperatures over the range 520 to 655°C for a period of 30 minutes. The hardness values were then re-checked and a graph constructed of hardness as a function of heat treatment temperature.

The reduction in hardness generally follows a linear relationship with tempering temperature, so consequently any tempering temperature which causes a reduction in hardness can be defined. The maximum temperature reached by the bolt, either during manufacture or as a consequence of heat treatment during the course of the fire, can be estimated.

The graphs shown in Figure 13 indicate that the maximum temperatures reached by the bolts were as follows:

Bolt No.	Position	Maximum Temperature Reached during Manufacture or During Fire
34	In fire zone	515°C
36	Edge of fire zone	455°C
37	In fire zone	515°C
40	In fire zone	540°C
47	Well away from fire	475°C

Hence none of the bolts tested were heated to temperatures in excess of 540°C either during manufacture or during the course of the fire.

This conclusion is confirmed by visual observation made on bolt No. 40 which still had its hot-dipped galvanised layer intact. Zinc has a melting temperature of 419°C, and would be expected to vapourise when heated to a temperature in excess of this value.

9.4 Conclusions from Material Testing

This study has indicated that the mechanisms which caused distortion of the structure were a relatively low temperature phenomenon. While this information is most important in defining specific temperatures at specific places in the structure it should be recognised that:

a) The bolts may not reflect the beam temperature in that the plated connections locally involved a heavier mass of steel, and consequently would have been heated at a slower rate, although there would have been some tendency to establish equilibrium in the long-duration fire.

b) The Fire Brigade fire-fighting activities would have been expected to keep the structure cool, particularly in the later stages of the fire.

c) Localised hot spots could occur above windows in the hut.

10. BEHAVIOUR OF THE STRUCTURE

10.1 General

The purpose of this section is to compare the actual behaviour of the steel structure with that predicted in the design of the building and with that expected under the observed fire conditions.

Many variables influence the deformations occurring in a structural frame during a fire and they generally vary both in location and time. If the fire had been a controlled test, it would have been difficult to measure temperature, loads and deformations at enough points and times to characterise the behaviour. Hence, it is not possible to be precise in analysing the behaviour in an accidental building fire where no measurements at all have been taken during the fire. Nevertheless, the state of the structure after the fire can be used to appraise its behaviour.

The purpose of design for fire is generally to meet a regulatory performance standard aimed at life safety. The building should not collapse onto adjacent property and should remain stable for long enough to enable occupants to escape. Prevention of unpredictable collapse in the interest of the safety of fire fighters is also essential. Design of the structure to minimise property damage in the event of a fire is left to the discretion of the owners of the building in consultation with their insurance company and is outside the scope of building authority requirements. Hence, the Broadgate building performed in a totally satisfactory manner in that no part of the structure collapsed during or after the fire even though some columns had not yet been fire protected.

Comparison of the behaviour of the structure with that predicted in the fire design is not directly possible because the building was designed as a set of individual elements whereas in reality it behaved as a frame. Idealised end conditions were assumed for beams, trusses, columns and floors and the simple bolted beam connections were considered to have no rotational stiffness. Interaction between these elements would necessarily have produced large restraining forces due to differential heating, but this was not considered in the design. While these assumptions may be generally conservative for the design of the structure, a realistic assessment of structural performance ought to consider these effects.

Although none of the elements of the Broadgate building collapsed, some members may have transferred load to adjacent "cooler" parts of the structure. It is not easy, nor is it realistic to define which elements would have been considered to fail had they not been supported by the adjacent members. The methods for design for fire which are generally available to the structural engineer in documents such as BS 5950, Part 8 (3), are aimed at preventing structural failure of a member or frame. These design methods ensure that the load capacity in fire is greater than the applied load. They are only appropriate where the member has reached the failure point. Calculation of deformations during the fire is a much more complex procedure requiring non-linear iterative analysis techniques. The code provides no guidance on the intermediate conditions. Determination of residual deformations after cooling is a much more difficult exercise for which reliable input and verification data are not available at present.

It is clear that the structural members were not heavily loaded at the time of the fire. The floor above the fire was empty apart from a site hut towards the western end, and the other levels of the building were lightly loaded.

In Section 9 it was concluded that the general temperature level around the structural steelwork was unlikely to have been as high as often experienced in building fires and certainly much lower than furnace temperatures used in standard fire-resistance tests to BS476, Parts 20 and 21 (16, 17). However, local hot spots may have occurred.

The columns had not been protected between Levels 1 and 2 at the time of the fire, and only small fragments of fire protection were in place on the other members after the fire. The time at which this protection was lost from the beams and trusses is an important parameter which is unknown.

Restraint from the adjacent structure generally causes compression in a member during the early stages of a fire, leading to an increase in the forces applied to the member. However, as flexural yielding occurs, the restraints change to tensile (or catenary) forces. In steel or composite structures with "robust" connections, these restraints are beneficial in terms of the collapse of the member in fire and increase its fire resistance in comparison to an isolated member.

In design, members are considered in isolation (refer to BS 5950 : Part 8). Exclusion of restraint effects from the design process is understandable, as their consideration would massively complicate the design of almost every type of member constructed from any material. It would be necessary to consider many possible scenarios of hot and cold parts of the structure. Generally the simplified approach used for design would still be expected to produce safe structures.

It has been well established (21, 22) that hot-rolled structural steel suffers negligible reduction in its strength after a fire. The testing by Stangers has confirmed this, although as noted above, their testing concentrated on regions away from the fire. Visual inspection of the steelwork is then the main means of damage assessment. The photographs taken during the week after the fire have been used for this assessment.

The behaviour of five types of structural element, beams, trusses, columns, floors and connections, are discussed below.

Table 2 and Figure 7 show the locations of the largest deformations of the beams, trusses and columns as determined from the available photographs and from a survey of the Level 2 floor. It may be seen that the most significant damage occurred at the western end of the fire zone. This area is one of two regions which had significantly more load. It is also likely that this area was subject to higher temperatures than the eastern end. However, it is unlikely that this additional load was the main cause of the additional damage in this area.

As discussed in Section 9, the bolts in the connections at the columns which failed did not exceed a temperature of 540°C for any significant length of time during the fire, while samples taken from deformed truss members had not exceeded 600°C. Standard fire-resistance tests on similar connections conducted for British Steel have shown temperature differences between steel beams and bolts of about 100°C. In these fire tests the beams and connections were fire-protected and subjected to a severe furnace environment for a period of one hour. Hence the heating of the beams and bolts in the fire test was very much in the transient heating stage with large temperature differences possible, whereas the beams and bolts in the Broadgate fire were heated unprotected for a long time at a relatively low temperature and were therefore approaching steady-state conditions. It is therefore likely that the temperature difference between the bolts and the beams was less than 100°C in the Broadgate fire.

The following discussion therefore assumes that the steelwork was at a temperature of 600°C. Because the large voids formed by the trapezoidal deck profile between the beams and the slab were not filled with fire protection material, a uniform temperature of 600°C over the depth of the beams has been assumed in the analysis.

10.2 Behaviour of Beams

Table 3 shows calculated load ratios for the beams which showed the greatest midspan deflections, along with limiting temperatures required to cause failure at these loads according to BS 5950 : Part 8. It may be seen that these temperatures are well in excess of 600°C. This is to be expected since the beams did not necessarily reach the point of failure during the fire. It is evident that additional forces must have been experienced by the beams in order to reduce their limiting temperatures to 600°C. The following explanation is offered:

Any beam heated to 600°C will undergo significant longitudinal expansion if not restrained. Figure 14 shows possible stages in the behaviour of a beam up to failure. The end connections were generally double-angle cleats bolted over most of the depth of the web. These would have had little longitudinal stiffness compared with that of the expanding beam and would have deformed at the bolt holes. There was nominally 5mm to 10mm clearance between the end of each beam and the member it was framing into. It can be expected that the end of the beam would have come into direct bearing with the connecting member. In cases where the connecting member was a light truss member with no beam on the other side, there would have been little stiffness and little axial force generated. In cases where it was a column passing through the floor slab or a beam with another beam framing into the opposite side, the restraining stiffness would have been substantially higher and large axial forces would have been generated in the beam. Because of the negative moments generated at the ends, these forces may have been greatest in the bottom flange and eventually caused buckling of this flange and possibly in the adjacent web. This would not constitute member failure, however, as the formation of a "hinge" at this point would then return the member to the simply-supported case which is the case originally assumed for the design. As the beam continued to deflect there would be significant shortening due to its large curvature. Hence, the compressive axial force would reduce and eventually become a tensile force in order to support the beam in catenary action. Provided these tensile forces could be adequately transferred to the rest of the structure, the beam would be unlikely to collapse.

The comments regarding bottom flange buckling are supported by the photographic evidence. In all cases where the bottom flange is buckled, the beam is framing into a stiff member as described above. However, there are cases where a beam framing into a stiff member has not buckled.

A computer program was written to determine the moment-curvature response of a restrained composite beam at elevated temperature. Any cross-section may be analysed for given values of temperature in the steel beam and the concrete slab. The method is described in Appendix 1. Assuming that temperatures are uniform along the length of the beam, a known moment distribution along the beam may be converted to a curvature distribution which may then be integrated to obtain the deflection at every point along the beam. The moment distribution used assumes that negligible negative moment is carried in the ends of the beam when the bottom flange is buckled near the supports. The program also gives the strain at every point over the depth of the beam, and integration of this strain along the length produces the free axial expansion of the member. Restraint of this expansion can produce large axial forces in the member.

The moment-curvature method has been applied to Beam D3, which showed the greatest deflection/span ratio after the fire. This beam size was the same as 7 out of the 11 beams with the greatest deflection/span ratios. A uniform temperature of 600°C has been used in the steel, and a temperature gradient in the concrete above the decking has been used. The resulting moment-curvature curves are shown in Figure A3.

The photographs of Beam D3 show that the bottom flange has severely buckled near the ends of the beam. This indicates that at some stage during the fire the combination of negative end moments and compressive axial force has produced compressive stresses at the bottom of the beam which have exceeded the elevated temperature buckling capacity of the bottom flange and reduced the negative moment capacity of the beam. After this time the beam can be considered as simply supported with an axial load applied through the slab.

This is the worst case scenario for beams with some rotation stiffness at the ends, i.e. continuity. In cases where the negative moments generated by continuity are not sufficient to buckle the bottom flange of the beam, this continuity can be very beneficial. In fact, beams which are able to carry these negative moments will always fail at a much higher temperature than if they were simply-supported.

Axial force produced by restraint of axial expansion adds to the compressive stress in the bottom flange produced by the negative end moments and makes bottom flange buckling more likely. Further work is required to establish the circumstances under which this will occur. Where sections classified as "plastic" are used, the geometry of the section is such that large ductile rotations are guaranteed under the influence of moment without flange buckling occurring. The strain in the compressive flange exceeds ten times the yield strain before buckling occurs. The influence of additional axial force on this rotation capacity at elevated temperature may possibly be established analytically. Buckling seems less likely to occur where the beams framing into each side of a connection are of similar size.

Appendix 1 shows deflection calculations based on the postulated moment-curvature curves shown in Figure A3. The midspan deflection with no axial force is 74mm. With an assumed axial force of 100 kN applied near the top of the slab, the deflection is increased to 120mm and when the axial force is increased to 200 kN the deflection is 195mm.

The deflection of 195mm is sufficient to cause "plastic" deformation of the beam, and therefore on cooling a substantial residual deflection would be expected. This would explain the observed deformation of beam D3 if it had been heavily restrained by the surrounding structure during the fire.

The axial movement of the members may also be calculated using this approach. The outward movement in the slab at each end of the composite beam was calculated to be 5.6mm for a 100kN additional compressive force and - 4.5mm for a 200 kN compressive force.

If the whole floor slab was uniformly heated then this expansion could occur freely, but in practice the parts of the floor slab away from the seat of the fire will be much cooler and being very stiff will offer considerable restraint. This is illustrated in Figure 15. A rough estimate of the stiffness of a cool sector of slab at the edge of a hot region may be made by considering it as an axially loaded column of width equal to the effective slab width for a composite beam (1500mm). This gives an elastic stiffness of 1100 kN/mm.

Compatibility of movement between the cool slab and the heated beam suggests that there may be some slight movement of the supports, reducing the assumed restraint force of 200kN to about 160kN with a corresponding reduction in the mid span deflection to about 160mm (see Figure A4).

This approximate analysis may be repeated for other temperatures. In general, restraint forces decrease as the member heats up and becomes more flexible relative to the surrounding structure.

10.3 Behaviour of Trusses

There were two types of truss in the fire zone - type T2 and type TG2. Both types showed large deflections after the fire. Their configurations are shown in Figure 5. Although they were constructed to act compositely with the floor slab, in practice the difference in structural performance compared with non-composite construction is almost negligible. Assuming all members to be pin-jointed, the member forces are therefore easily calculated for the applied vertical loading. These forces, load capacities and expected critical temperatures are shown in Table 4 for truss CD4 which is a type T2 truss. Similar results would be obtained for trusses CD5 and CD6 which are type TG2 trusses.

From Table 4 it is apparent that, if these trusses reached temperatures of about 600°C, they would not be expected to show the observed buckling type deformations. These deformations have apparently resulted from the existence of significant rotational fixity in the joints combined with differential heating in different parts of individual members and differential heating between adjacent members. This differential heating may have been caused by uneven fire attack and by differences in thermal mass between light and heavy members.

10.4 Behaviour of Columns

Loads and load ratios for the C-line and E-line columns which showed severe deformations and for the intervening much larger D-line columns which did not deform significantly are shown in Table 5.

It can be seen that the columns would not be expected to deform excessively simply under the applied loads if their temperatures were only around 600°C. Again it seems likely that the members have received substantial additional load due to restraint of thermal expansion. This restraint has come from the moment frame which spans east-west on Levels 8 and 9 and from the connections to the beams spanning north-south, which although they are not designed to transfer moment do in fact have considerable rotational stiffness when summed over the height of the building. Fortunately some useful information is available on the stiffness of the frame from the measurements taken during the jacking operation which was done after the fire to restore the original floor levels.

The magnitude of the additional force developed in any column depends on its temperature differential relative to the adjacent columns. In the extreme case where all columns at a floor level are heated at the same rate, no restraining forces are developed. At the other extreme where only one column is heated the greatest restraining force is produced. This is illustrated in Figure 16.

The effect of column restraint has been analysed by Bennetts, Goh, O'Meagher and Thomas (28) for three typical steel-framed buildings. The worst case of a single column being heated has been considered and conservative values of connection stiffness have been calculated using elastic theory. The type of beam-column connection considered is a web side-plate, consisting of a vertical plate welded to the column and connected by bolts through the web of the beam. This study showed that, after making realistic allowances for non-uniform temperature over the column height and some plastic deformation, the effect of the restraining force was to reduce the failure temperature by 30°C for a ten storey building and 80°C for a twenty storey building. While these results show only a small effect of restraint, the effect may be more significant for buildings with greater connection stiffnesses.

A calculation method based on one column being heated to a greater temperature than its neighbours has been made. Knowing the applied load and the load capacity, the additional restraint force required to cause column failure is determined. For a given temperature differential, the free thermal expansion is calculated. Hence, an equivalent stiffness to produce the required additional force is obtained. This equivalent stiffness is a function of the stiffness of the column being analysed and the stiffness of the rest of the structure. Hence the required stiffness of the adjacent parts of the structure can be obtained.

The calculation method has been applied to the C-line, D-line and E-line columns and the results are shown in Table 6. It may be seen that if the stiffness of the adjacent structure is greater than 100-300 kN/mm then the C-line and E-line columns may possibly be failed by thermal restraint effects. It should be remembered, however, that this is the worst case scenario in which adjacent columns are cool. It may be seen that the required stiffness values for the D-line columns are substantially greater than for the C-line and E-line columns.

For comparison, the following stiffness values have been taken from the Tony Gee report on the jacking operation (29):

Column C33, jacking up	: k =	1400 kN/mm
Jack release	: k =	500 kN/mm
Column C34, jacking up	: k =	100 kN/mm
Jack release	: k =	27 kN/mm

The large differences between jacking up and "release" stiffnesses are due to frictional effects within the structure. For comparison with the value for column C34, SOM have calculated a stiffness of approximately 20 kN/mm using elastic frame analysis and assumed connection stiffnesses.

Although there is much uncertainty about the actual stiffness of the structure, it is apparent that the stiffness is sufficient to increase the loads applied to the columns. In the case of the columns that deformed excessively these addition of forces could have been very high.

Nevertheless, it is important to recognise that despite these increased forces due to differential heating, the surrounding structure would also help to "support" the columns once they had reached their maximum capacity. This re-distribution of forces due to the development of alternative load paths is a very important beneficial factor in fire conditions. Therefore, the final "collapse" of the structure would occur at much higher temperatures than indicated by this assessment of the performance of a single column. This is analogous to the restraint effect on beams in fire.

It was noted in fact during the replacement of the damaged columns that they were carrying substantially less than they would have carried in an undeformed structure. S.O.M. have supplied the following information which clearly shows this redistribution of load from a badly deformed column to its neighbours.

Column	C32	C33	C34	E33	E34
Load required to open Level 2 splice (kN)	760	720	500	450	520
Calculated load based on actual applied floor loads (kN)	1330	1320	1420	1020	1500
Load carried / Load calculated (%)	57	55	35	44	35

10.5 Behaviour of the Floor System

Fire Damage

After the fire the following features were observed:

1. The profile of the floor generally followed that of the beams without substantial additional deflection between these supporting members.

2. There were three major cracks of width exceeding 5mm in the floor slab. The largest was 10m in length and produced a 10mm vertical step in the slab surface. It ran north-south alongside beams D2, D3 and D4. The second was 6m in length and ran east-west alongside beam AC7. The third was 2m in length and ran east-west alongside beam EF5. These cracks were all at the boundaries of large floor regions which had deflected substantially due to sagging of the supporting beams and trusses. Necking failure of the slab reinforcing mesh was observed at the largest crack. It was generally noted, however, that the slab contained few cracks compared with those which were normally observed in a fire resistance test.

3. The deck had become debonded from the slab in a number of areas as a consequence of thermal restraint at the ends of the deck due to the surrounding structure.

4. The deck had pulled away from the top flange of supporting beams in isolated areas despite the use of shot-fired fixings. This phenomenon was only observed on the smallest sections, where any relative movement between the narrow top flange and the deck was obviously most likely to cause the deck to pull off the flange.

5. The thermal insulation performance of the slab was generally good with no major evidence of failure of its insulation characteristics, although the plastic membrane liner between the slab and the atrium marble floor above did show some degradation.

6. There was no evidence of any loss of overall integrity of the slab/steel deck in the area.

7. There was some evidence of gross deformation of the steel deck whereby the wide ribs of the trapezoidal profile had become inverted and ballooned downwards between the concrete-filled troughs. This was thought to be due

to pressure built up as a consequence of steam release from the concrete during the heating process. This phenomenon was only observed in isolated areas, presumably where routes were not available for steam release.

Debonding of the steel deck occurred over most of the region affected by fire, and this is to be expected because of the large temperature difference quickly set up between the thin steel sheet and the concrete. As far as the performance of the floor is concerned, however, this is of little consequence, since the deck is considered to contribute relatively little strength of the floor in fire. It is possible that the air gap formed between the deck and the slab provides some insulation for the slab and the presence of the deck stops direct flame impingement on the concrete, both of which would help to avoid spalling of the concrete. The stability of the floor is provided by the reinforcement within the concrete, which is kept relatively cool by the concrete below it and which operates above the stress levels utilised in normal-temperature design to provide the required load capacity in fire.

This debonding of the deck should be considered in the context that the severe nature of the fire and the high moisture content of the "young" concrete will both have contributed to a far greater tendency for debonding than in a "dry" slab. The important conclusion of this investigation was that the floor did not fail and the upper slab surface did not deform significantly relevant to the beams.

Reinstatement of Fire Damaged Areas

The areas of floor to be replaced were decided following extensive testing to establish:

1. Any reduction in concrete strength.
2. Any debonding between the deck and the concrete.
3. Any degradation of the galvanised protective layer of the deck.
4. Straightness of the supporting beam structure.

The acceptability levels for each parameter were considered in depth by the design team and in areas of doubt the floor system was replaced. A major factor which influenced the extent of deck replacement was "convenience", i.e. consideration of where the existing butt joints were within the deck and the design condition in the reinstated structure with respect to the positions of induced discontinuities. In any event the reinstated design had to take into account the normal parameters with respect to design conditions and laps, etc.

Cleaning of the Deck

The removal of the corrosive sooty deposits did present some problems. These deposits were eventually removed by manual scrubbing with an aqueous solution containing a proprietary product which was alkaline in nature, (sodium metasilicate, fatty alcohol ethoxylale, anionic surfactant 2 Butoxy-ethanol pH = 12.5-13.0) which would neutralise any acidic residues. There was concern after cleaning that acidic sooty deposits had penetrated the lap joints in the metal deck. Testing proved that low concentrations existed in a limited number of parts of the structure. This contamination was sealed in place using a silicone based sealant.

10.6 Behaviour of Connections

The connections showed evidence of the development of large forces and moments built up through restraint of thermal expansion of the various elements. This is consistent with the hypothesis that the bulk of the deformation observed in the frame can be accounted for by restraint rather than by high temperatures being attained.

An important observation from the photographs is that few of the bolts fractured. Because the bolt material has a higher strength and hardness than the adjacent structural steel elements, bolts that were put into bearing deformed the holes in the connecting member rather than being heavily deformed themselves. Thus the required axial movements and rotations of the beam ends could be accommodated. This was possible in the bulk of the connections used which consisted of double angles bolted through the beam web and bolted to the supporting member. Only in the cases where bearing of the bolts was not possible did any bolt fracture occur. A few of the connections consisted of an end plate welded to the web of the beam and bolted to the column. In this case relative movement between the end plate and the web was not possible. and after a small amount of deformation available in the end plate itself the bolts connecting to the column were put into tension. Because of the high axial stiffness of this arrangement in a few cases the tensile capacity of the bolts was exceeded.

The following paragraph has been extracted from the S.O.M. report:

> "There were two beams in the fire zone with flexible end plate connections. The connection on one end of each of these beams failed. Two of the bolts fractured in one connection, and the plate fractured full length on the other. Both beams were 254 x 102 framed between columns. The beams were reported to have been fireproofed. It would appear that the failure occurred as a result of the tension forces perpendicular to the connection that developed as the member cooled after the fire. The plate in one case and the bolts in the other did not have enough deformation capacity to accommodate the strains that occurred. It is significant that the only two beams with flexible end plates each had a connection failure, while there was no failure of any other connection."

Tensile failure of the connections on cooling is unlikely to occur because fracture of over-stressed bolts will partly relieve the tensile forces on the connection. A study has been made of the effect of fracture of half of the bolts in the connection on cooling. It is concluded that there is sufficient load-carrying capacity of the remaining bolts in shear to continue to support the loads (at the fire limit state) once the structure has cooled down. This analysis took into account the effect of elevated temperatures on the bolt strengths (assumed to be as high as $600^{o}C$) and the inherent partial safety factors used in the design of bolts in normal conditions.

Therefore the observed failure of certain bolts should not be considered out of context as this effect will not result in catastrophic failure of the member. It does suggest that the other connected parts such as welds should be of sufficient size that they do not fail prematurely.

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JUNE, 1991

ADDENDUM 1

FIRE PRECAUTIONS DURING THE CONSTRUCTION PHASE

ADDENDUM 1

FIRE PRECAUTIONS DURING THE CONSTRUCTION PHASE

The damage caused as a consequence of the Broadgate fire and the potential consequences of such a fire during the day-time on the life safety of the occupants of the building has highlighted the necessity to consider appropriate fire precautions during the construction phase.

The purpose of this addendum to the report is to highlight some of the aspects of fire precautions which should be considered.

Legislation

The principle legislation concerned with fire safety in buildings, i.e. the Fire Precautions Act of 1971, the Fire Safety and Places of Sport Act 1987, and the Building Regulations 1985, cannot be directly applied to buildings under construction. The main legislation which can be used to enforce fire precautions during construction is the Health & Safety at Work, Etc. Act 1974, which is concerned with securing the health, safety and welfare of persons at work, and with protecting people who are not at work from risks to their health and safety arising from work activities.

The principle enforcing authority for this legislation is the Health and Safety Executive. The employer has a duty to keep his work place in a safe condition without risk to health and it should include maintaining safe means of access and egress, providing training, supervision and information to ensure health and safety. Consequently employees must be conversant with fire drills and fire precautions.

In the case of temporary buildings, legislation listed in the Fire Certificates (Special Premises) Regulations 1976 can be applied to:

Any building, or part of a building, which either:

a) is constructed for temporary occupation for the purpose of building operations or works of engineering construction; or

b) is in existence at the first commencement there of any further such operations or works;

and which is used for any process or work ancillary to any such operations or works.

A fire certificate will not be required if either:

i) not more than 20 persons are employed at any one time in the building or part of the building; or

ii) not more than 10 persons are employed at any one time elsewhere than on the ground floor of the building or part of the building.

The Regulations are enforced by the Health and Safety Executive who will issue fire certificates as appropriate.

Objectives

The objectives of any fire precautions during the construction phase should be related to the safety of the construction workers in all parts of the building, and the safety of other persons in and around the building.

Fire precautions should also be associated with property protection and damage to building work which has been completed.

Site management should develop its own procedures in guidance documents which are tailored to the specific requirements of the site to ensure that the above objectives are satisfied.

Site Access

Consideration should be given to access facilities for the fire brigade bearing in mind width, height and weight limit restrictions of fire appliance vehicles, recognising the positions of the operational hydrants for water supplies.

Position of Office Accommodation and Means of Escape

It is important that "means of escape" be provided for all occupants of the building at all times and consequently travel distances and routes both within the hut and within the floor space of the building should meet current Code requirements. If temporary office accommodation is required it should be located in a convenient position for fire fighting operations and means of escape. Clear defined routes should be provided through temporarily stored goods and equipment. The temporary accommodation should be small such that the possibility of fire spread to adjacent accommodation or other stored goods is remote.

There should be easy access to the temporary accommodation from staircases which should be located close to windows to allow release of smoke. Locations which could provide an easy route for fire spread and smoke spread through the building, i.e. within atria, etc. should be avoided.

Building Construction Works

To prevent fire and smoke spread throughout the building, doors to staircases, lift shafts and service risers should be installed as soon as the temporary accommodation is occupied. Similarly, openings through ducts and cavities between curtain wall and floor slabs should be protected as soon as the curtain walling is completed.

The form of construction of the huts should be considered, and in particular the surface spread of flame characteristics and fire resistance requirements if any.

Facilities

As soon as the temporary accommodation is erected any dry risers which are to be installed in the building should be fitted and checked to ensure that they are working to required standards. A modular fire detection and alarm system should be installed within the temporary office accommodation in order to ensure that the earliest possible warning of the existence of fire be given to the occupants of the structure.

First aid fire fighting equipment (extinguishers and hose reels) should be readily available.

The possibility of using sprinkler systems within temporary accommodation requires careful consideration, bearing in mind the practicality of adopting such systems and the potential risk.

Developments are under way in the aircraft industry to produce sprinkler systems which are most effective in controlling fires while using small quantities of water. It may be possible to adapt such technologies to temporary site accommodation by using a small "in situ" water tank incorporated into the design and structure.

Emergency lighting systems should be installed in escape routes.

Signs relating to emergency facilities/equipment should be accurate. Temporary hoardings should not prevent access to these facilities or equipment.

Housekeeping

It is the responsibility of the construction managers to ensure that good housekeeping is maintained, and the collection of rubbish from various parts of the building avoided.

Management and Communication

It is the responsibility of site management to ensure that the fire precaution measures proposed are pursued by all concerned. Staff training and drills should be implemented on a regular basis. Informative poster notices regarding the importance of fire precautions should be displayed throughout the building. Site management should also liaise with fire brigade personnel to ensure that they are familiar with both the layout of the building and the fire precaution measures which have been implemented.

Security patrols should be carried out at all times, and in particular at situations where the building is unoccupied. In the event of a fire emergency the security personnel should play an integral role in the early stage of the fire incident to inform attending crews of the status of the facilities provided and which risers and staircases are in operation, etc.

Guidance Documents

Additional comprehensive guidance on fire precaution measures which should be adopted during the construction phase are required as a matter of urgency.

Three distinct phases can be identified during construction as follows:

Building Construction
Fit Out Phase
Partial Occupation

The various organisations involved, i.e. construction managers, tenant and landlord must assume different responsibilities for fire precautions at different stages.

Clear guidelines are therefore required on requirements, practices and enforcement procedures. The only publication available at this stage "Standard Fire Precautions - P5" has been published by the P.S.A. at the Department of the Environment and deals with measures to be taken by contractors engaged on building and emergency work and maintenance for the D. of E.

APPENDIX 1

DEFLECTION AND ELONGATION CALCULATIONS FOR SIMPLY SUPPORTED BEAM WITH AXIAL FORCE

APPENDIX 1

MOMENT-CURVATURE ANALYSIS OF COMPOSITE BEAMS

Moment-Curvature Method:

Figure A1 shows schematically the application of the moment-curvature method to composite beams in fire. The cross-section is divided over its height into small regions which are horizontal strips of steel or concrete. Each region has an associated temperature, strain and stress. The main steps are:

(a) Input the temperature in each cross-section region.

(b) Calculate the free thermal strain in each region from its temperature.

(c) Apply trial values of total strain as a plane section. This plane is defined by the curvature on the cross-section and the strain at the bottom of the section.

(d) Calculate the stress-inducing strain as the difference between total strain and thermal strain in each region.

(e) Calculate the stress in each region from its stress-inducing strain and temperature.

(f) Multiply all stresses by their respective areas and sum to obtain cross-section total force and total moment.

(g) Keeping curvature constant, vary the total strain until total force equals applied axial force. Because the stress-strain curve for concrete is not monotonic, some care is required to ensure that the procedure iterates to the correct solution.

(h) Repeat this process for any desired values of curvature and time of fire exposure.

The stress-strain-temperature and thermal expansion relationships used for reinforcing steel and concrete in this procedure are those defined in EC4: Part 10: 1990(30).

Input Data and Results

For the moment-curvature runs conducted, the following input parameters have been used:

Beam size: 254 x 102 x 25 UB

Overall slab depth: 130mm

Steel deck rib height: 60mm

Slab width: 1500mm

Slab reinforcement: A142 mesh, 30mm top cover

Steel beam yield stress: 400 N/mm²

Concrete compressive strength (as per EC4): 40 N/mm²

Reinforcement yield stress: 450 N/mm²

Stress-strain-temperature relationships as per EC4: Part 10: 1990

Steel beam teperature uniform over depth of beam

Slab temperature profile above deck rib representative of fully-exposed slab face at 0.2hr of standard fire exposure, as follows:

$$T(x) = 710.4\ e^{-\frac{x}{24.53}}$$

where x = distance from top of rib (mm)

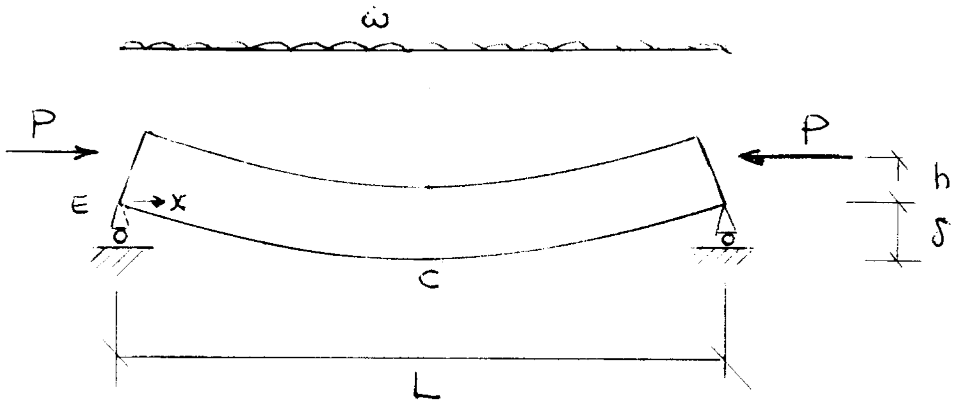
$T(x)$ = concrete temperature (°C)

Reinforcement temperature equal to concrete temperature at mid-height of reinforcement.

The moment curvature results obtained are shown in Figure A3 for 600°C steel temperature. and Figure 17 for 300°C steel temperature. A sample solution at 600°C with 100 kN axial force is shown in Table A2.

DEFLECTION AND ELONGATION CALCULATIONS FOR SIMPLY SUPPORTED BEAM WITH AXIAL FORCE

General Equations



Taking moments about bottom of beam:

Centre: $M_C = \dfrac{\omega l^2}{8} + P(\delta + h)$

End: $M_E = Ph$

Put ϕ = beam curvature

$\overline{\phi}$ = average curvature over length $= \dfrac{1}{L}\int_0^L \phi(x)\ dx$

ϵ_b = strain at bottom of beam

$\overline{\epsilon_b}$ = average bottom strain over length $= \dfrac{1}{L}\int_0^L \epsilon_b\ dx$

Δ = axial elongation at each end of height h

Δ_e = elongation at height h from summing strains along $\dfrac{L}{2}$

Δ_s = shortening due to beam curvature

For parabolic $M(x)$ and linear $M(\phi)$ and linear $\epsilon_b(\phi)$:

$$\overline{\phi} = \frac{2}{3}\phi_C + \frac{1}{3}\phi_E$$

$$\overline{\epsilon_b} = \frac{2}{3}\epsilon_{bC} + \frac{1}{3}\epsilon_{bE}$$

For constant curvature, ie. $\phi(x) = \overline{\phi}$:

$$\delta = \frac{\overline{\phi}L^2}{8}$$

$$\Delta_e = (\overline{\epsilon_b} - \overline{\phi}h)\frac{L}{2}$$

$$\Delta_s = \frac{(\phi L)^2}{24}\cdot\frac{L}{2}$$

Beam D3, No Axial Force, 600°C

Beam supports	3.0m floor @ 1.77 kN/m²	=	5.31 kNm
	1.5m hut @ 1.5 kN/m²	=	2.25
	1.5m marble @ 1.75 kN/m²	=	2.63
	Self-weight	=	0.25
	ω	=	10.44 kNm

$L = 6.0\text{m} \qquad \dfrac{\omega L^2}{8} = 47.0 \text{ kNm}$

$P = 0: \qquad M_C = 47.0, \quad M_E = 0$

From Figure A3: $\phi_C = 22.5 \text{ km}^{-1}, \ \phi_E = 4.2\text{km}^{-1}$ (at 600°C steel temperature)

Hence, $\bar{\phi} = \dfrac{2}{3} \ . \ 22.5 + \dfrac{1}{3} \ . \ 4.2 = 16.4$

$$\delta = 16.4 \ . \ \frac{6^2}{8} = 73.8 \text{ mm}$$

For Figure A3: $\epsilon_{bC} = 1.20\% \ , \ \epsilon_{bE} = 0.84\%$

$$\bar{\epsilon_b} = \frac{2}{3} \ . \ 1.20 + \frac{1}{3} \ . \ 0.84 = 1.08\%$$

For $h = 0.380\text{m}, \ \Delta_e = (1.08.10^{-2} - 16.4 \ . \ 0.380.10^{-3}) \ . \ 3000 = 13.7 \text{ mm}$

$$\Delta_s = \frac{(16.4 \ . \ 6.0.10^{-3})^2}{24} \ . \ 3000 = 1.2 \text{ mm}$$

$$\Delta = \Delta_e - \Delta_s = 12.5 \text{ mm}$$

Solution: $\delta = 73.8\text{mm}, \ \Delta = 12.5\text{mm}$

<u>Beam D3, 100kN Axial Force, 600°C</u>

Apply a 100kN axial force at 380mm from the bottom of the composite beam (or 7mm from the top of the concrete slab).

Iterative solution - try δ = 100mm:

$$M_C = 47.0 + 100(0.100 + 0.380) = 95.0 \text{ kNm}$$

$$M_E = 100 \, . \, 0.380 = 38.0 \text{ kNm}$$

From Figure A3: $\phi_C = 33.3 \text{ km}^{-1}$, $\phi_E = 11.1 \text{ km}^{-1}$

$$\bar{\phi} = \frac{2}{3} \, . \, 33.3 + \frac{1}{3} \, . \, 11.1 = 25.9 \text{ km}^{-1}$$

$$\delta = 25.9 \, . \, \frac{6^2}{8} = 116.5 \text{ mm}$$

$$M_C = 47.0 + 100(0.1165 + 0.380) = 96.7 \text{ kNm}$$

From Figure A3: $\phi_C = 34.4 \text{ km}^{-1}$

$$\bar{\phi} = \frac{2}{3} \, . \, 34.4 + \frac{1}{3} \, . \, 11.1 = 26.6 \text{ km}^{-1}$$

$$\delta = 26.6 \, . \, \frac{6^2}{8} = 119.7 \text{ mm}$$

$$M_C = 97.0 \text{ kNm}$$

$$\phi_C = 34.5$$

$$\bar{\phi} = 26.7$$

$$\delta = 120.2 \text{ mm} \quad \text{OK}$$

From Figure A3: at $\phi_C = 34.5$, $\epsilon_{bC} = 1.51\%$

at $\phi_E = 11.1$, $\epsilon_{bE} = 0.90\%$

$$\bar{\epsilon}_b = \frac{2}{3} \, . \, 1.51 + \frac{1}{3} \, . \, 0.90 = 1.31\%$$

$$\Delta_e = (1.31.10^{-2} - 26.7 \, . \, 0.380.10^{-3}) \, . \, 3000 = 8.8 \text{ mm}$$

$$\Delta_s = \frac{(26.7 \, . \, 6.0.10^{-3})^2}{24} \, . \, 3000 = 3.2 \text{ mm}$$

$$\Delta = 8.8 - 3.2 = 5.6 \text{ mm}$$

<u>Solution</u>: δ = 120mm, Δ= 5.6mm

Beam D3, 200kN Axial Force, 600°C

Apply a 200kN axial force at 380mm from the bottom of the beam.

Try δ = 140mm:

$$M_C = 47.0 + 200(0.140 + 0.380) = 151.0 \text{ kNm}$$

$$M_E = 200 \,.\, 0.380 = 76.0 \text{ kNm}$$

From Figure A3: $\phi_C = 50.6 \text{ km}^{-1}$, $\phi_E = 15.7 \text{ km}^{-1}$

$$\bar{\phi} = \frac{2}{3} \,.\, 50.6 + \frac{1}{3} \,.\, 15.7 = 39.0 \text{ km}^{-1}$$

$$\delta = 39.0 \,.\, \frac{6^2}{8} = 175.4 \text{ mm}$$

$$M_C = 158.1 \text{ kNm}$$

$$\phi_C = 55.1 \text{ km}^{-1}$$

After three more iterations, converges at:

$$M_C = 161.9 \text{ kNm}$$

$$\phi_C = 57.2 \text{ km}^{-1}$$

$$\bar{\phi} = 43.4 \text{ km}^{-1}$$

$$\delta = 195.1 \text{ mm}$$

From Figure A3: at $\phi_C = 57.2$, $\epsilon_{bC} = 2.20\%$

at $\phi_E = 15.7$, $\epsilon_{bE} = 0.95\%$

$$\bar{\epsilon}_b = \frac{2}{3} \,.\, 2.20 + \frac{1}{3} \,.\, 0.95 = 1.78\%$$

$$\Delta_e = (1.78.10^{-2} - 43.4 \,.\, 0.380.10^{-3}) \,.\, 3000 = 4.0 \text{ mm}$$

$$\Delta_s = \frac{(43.4 \,.\, 6.0.10^{-3})^2}{24} \,.\, 3000 = 8.5 \text{ mm}$$

$$\Delta = 4.0 - 8.5 = -4.5 \text{ mm (i.e shortening)}$$

Solution: δ = 195mm, Δ = -4.5mm

This solution is not practical since overall shortening of the beam at h=380mm would induce tensile restraining forces.

Estimate Applied Axial Force, 600°C

Since the slab is extremely stiff in comparison with the beam, very little axial movement will be possible. The axial force applied may be estimated by interpolating for the value of P which gives $\Delta = 0$

For $P = 0$, $\Delta = +12.5$mm, $\delta = 73.8$mm

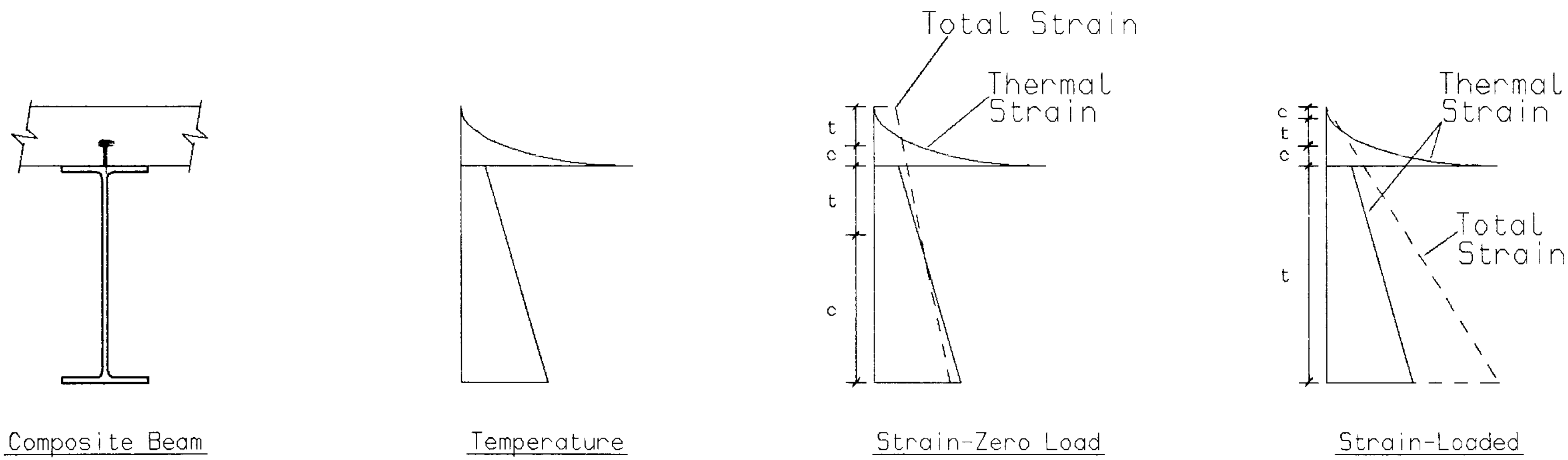
$P = 100$kN, $\Delta = +5.6$mm, $\delta = 120$mm

$P = 200$kN, $\Delta = -4.5$mm, $\delta = 195$mm

Hence, $P \simeq 159$ kN for $\Delta = 0$ (see Figure A4)

P must be slightly less than this value for Δ slightly greater than zero.

Solution: $P \simeq 157$ kN, $\Delta \simeq 0.2$mm, $\delta \simeq 160$mm



Stress-Inducing Strain = Total Stain - Thermal Strain
c = Compressive Stress-Inducing Strain
t = Tensile Stess-Inducing Strain

Figure A1: Moment-Curvature Analysis of Composite Beams

AXIAL FORCE = 100 kN
TIME= .2 HOURS
CURVATURE = 40 km-1

	HEIGHT (mm)	TEMP (degC)	STRAIN (millistrain) THERMAL	TOTAL	STRESS-INDUCING	STRESS (MPa)	FORCE (kN)
RE	353.0	163.7	1.9	2.6	0.6	122.8	17.4
SL	385.8	42.9	0.3	1.3	1.0	0.0	0.0
SL	383.5	47.2	0.3	1.4	1.1	0.0	0.0
SL	381.2	51.9	0.4	1.5	1.1	0.0	0.0
SL	378.8	57.1	0.4	1.5	1.1	0.0	0.0
SL	376.5	62.8	0.5	1.6	1.1	0.0	0.0
SL	374.2	69.1	0.6	1.7	1.1	0.0	0.0
SL	371.8	76.0	0.7	1.8	1.1	0.0	0.0
SL	369.5	83.5	0.8	1.9	1.1	0.0	0.0
SL	367.2	91.9	0.9	2.0	1.1	0.0	0.0
SL	364.8	101.0	1.1	2.1	1.0	0.0	0.0
SL	362.5	111.1	1.2	2.2	1.0	0.0	0.0
SL	360.2	122.2	1.4	2.3	0.9	0.0	0.0
SL	357.8	134.4	1.5	2.4	0.9	0.0	0.0
SL	355.5	147.8	1.7	2.5	0.8	0.0	0.0
SL*	353.2	162.6	1.9	2.6	0.6	0.0	0.0
SL	350.8	178.8	2.2	2.7	0.5	0.0	0.0
SL	348.5	196.7	2.4	2.8	0.4	0.0	0.0
SL	346.2	216.3	2.7	2.9	0.2	0.0	0.0
SL	343.8	237.9	3.0	2.9	-0.0	-0.3	-1.2
SL	341.5	261.6	3.3	3.0	-0.3	-2.6	-9.1
SL	339.2	287.7	3.7	3.1	-0.5	-4.8	-16.8
SL	336.8	316.5	4.1	3.2	-0.9	-6.8	-23.9
SL	334.5	348.1	4.5	3.3	-1.2	-8.6	-30.0
SL	332.2	382.8	5.0	3.4	-1.6	-10.1	-35.3
SL	329.8	421.0	5.5	3.5	-2.0	-11.0	-38.5
SL	327.5	463.0	6.1	3.6	-2.5	-11.2	-39.3
SL	325.2	509.2	6.8	3.7	-3.1	-10.9	-38.2
SL	322.8	560.1	7.5	3.8	-3.7	-9.9	-34.5
SL	320.5	616.0	8.3	3.9	-4.4	-8.6	-30.2
SL	318.2	677.4	9.1	4.0	-5.2	-7.4	-25.8
TF	256.6	600.0	8.0	6.4	-1.6	-90.2	-6.4
TF	255.9	600.0	8.0	6.5	-1.6	-89.5	-6.3
TF	255.3	600.0	8.0	6.5	-1.6	-88.7	-6.3
TF	254.6	600.0	8.0	6.5	-1.5	-87.9	-6.2
TF	253.9	600.0	8.0	6.5	-1.5	-87.1	-6.2
TF	253.1	600.0	8.0	6.6	-1.5	-86.2	-6.1
TF	252.4	600.0	8.0	6.6	-1.4	-85.4	-6.0
TF	251.8	600.0	8.0	6.6	-1.4	-84.4	-6.0
TF	251.1	600.0	8.0	6.7	-1.4	-83.5	-5.9
TF	250.4	600.0	8.0	6.7	-1.4	-82.5	-5.8
TF	249.6	600.0	8.0	6.7	-1.3	-81.5	-5.8
TF	248.9	600.0	8.0	6.7	-1.3	-80.4	-5.7
WB	244.0	600.0	8.0	6.9	-1.1	-70.4	-4.0
WB	234.7	600.0	8.0	7.3	-0.7	-46.9	-2.6
WB	225.5	600.0	8.0	7.7	-0.4	-23.4	-1.3
WB	216.3	600.0	8.0	8.1	0.0	0.1	0.0
WB	207.0	600.0	8.0	8.4	0.4	23.5	1.3
WB	197.8	600.0	8.0	8.8	0.7	47.0	2.7
WB	188.6	600.0	8.0	9.2	1.1	70.5	4.0
WB	179.3	600.0	8.0	9.5	1.5	86.4	4.9
WB	170.1	600.0	8.0	9.9	1.8	96.0	5.4
WB	160.8	600.0	8.0	10.3	2.2	103.4	5.8
WB	151.6	600.0	8.0	10.6	2.6	109.7	6.2
WB	142.4	600.0	8.0	11.0	3.0	115.2	6.5
WB	133.1	600.0	8.0	11.4	3.3	120.1	6.8
WB	123.9	600.0	8.0	11.7	3.7	124.6	7.0
WB	114.6	600.0	8.0	12.1	4.1	128.6	7.2
WB	105.4	600.0	8.0	12.5	4.4	132.4	7.5
WB	96.2	600.0	8.0	12.9	4.8	135.9	7.7
WB	86.9	600.0	8.0	13.2	5.2	139.2	7.8
WB	77.7	600.0	8.0	13.6	5.5	142.2	8.0
WB	68.4	600.0	8.0	14.0	5.9	145.1	8.2
WB	59.2	600.0	8.0	14.3	6.3	147.9	8.3
WB	50.0	600.0	8.0	14.7	6.7	150.4	8.5
WB	40.7	600.0	8.0	15.1	7.0	152.9	8.6
WB	31.5	600.0	8.0	15.4	7.4	155.2	8.7
WB	22.3	600.0	8.0	15.8	7.8	157.4	8.9
WB	13.0	600.0	8.0	16.2	8.1	159.4	9.0
BF	8.0	600.0	8.0	16.4	8.3	160.5	11.3
BF	7.3	600.0	8.0	16.4	8.4	160.7	11.4
BF	6.6	600.0	8.0	16.4	8.4	160.8	11.4
BF	5.9	600.0	8.0	16.5	8.4	161.0	11.4
BF	5.3	600.0	8.0	16.5	8.4	161.1	11.4
BF	4.5	600.0	8.0	16.5	8.5	161.3	11.4
BF	3.8	600.0	8.0	16.5	8.5	161.4	11.4
BF	3.1	600.0	8.0	16.6	8.5	161.6	11.4
BF	2.4	600.0	8.0	16.6	8.6	161.7	11.4
BF	1.7	600.0	8.0	16.6	8.6	161.8	11.4
BF	1.0	600.0	8.0	16.7	8.6	162.0	11.5
BF	0.3	600.0	8.0	16.7	8.6	162.1	11.5

* = concrete elements containing reinforcement
STRAIN AT BOTTOM = 16.70155
MOMENT CAPACITY = 105.5516 kN-m

Table A2: Sample Moment-Curvature Solution

Discuss me ...

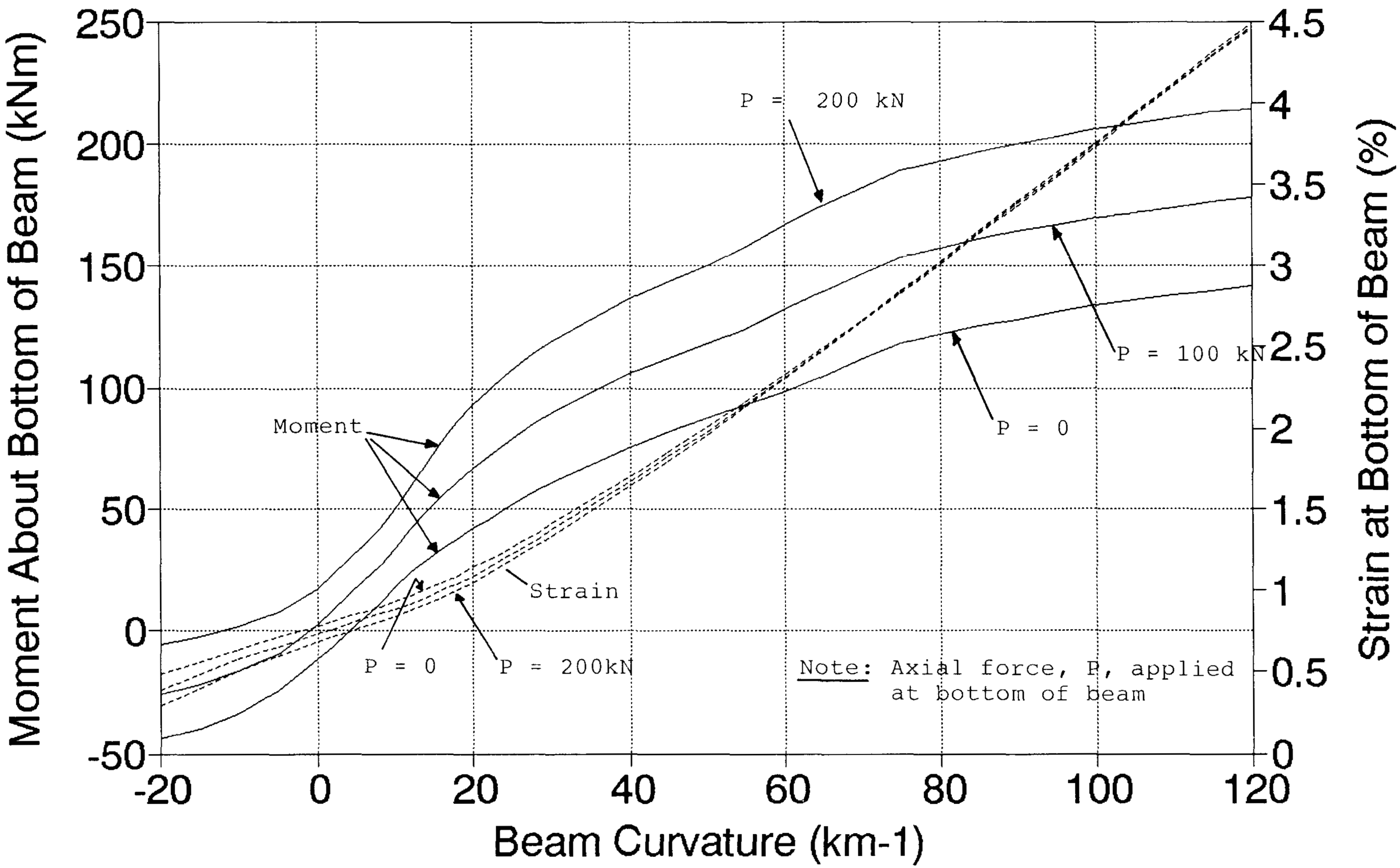


Figure A3: Moment-Curvature Curves for Beam D3 at 600 (deg C)

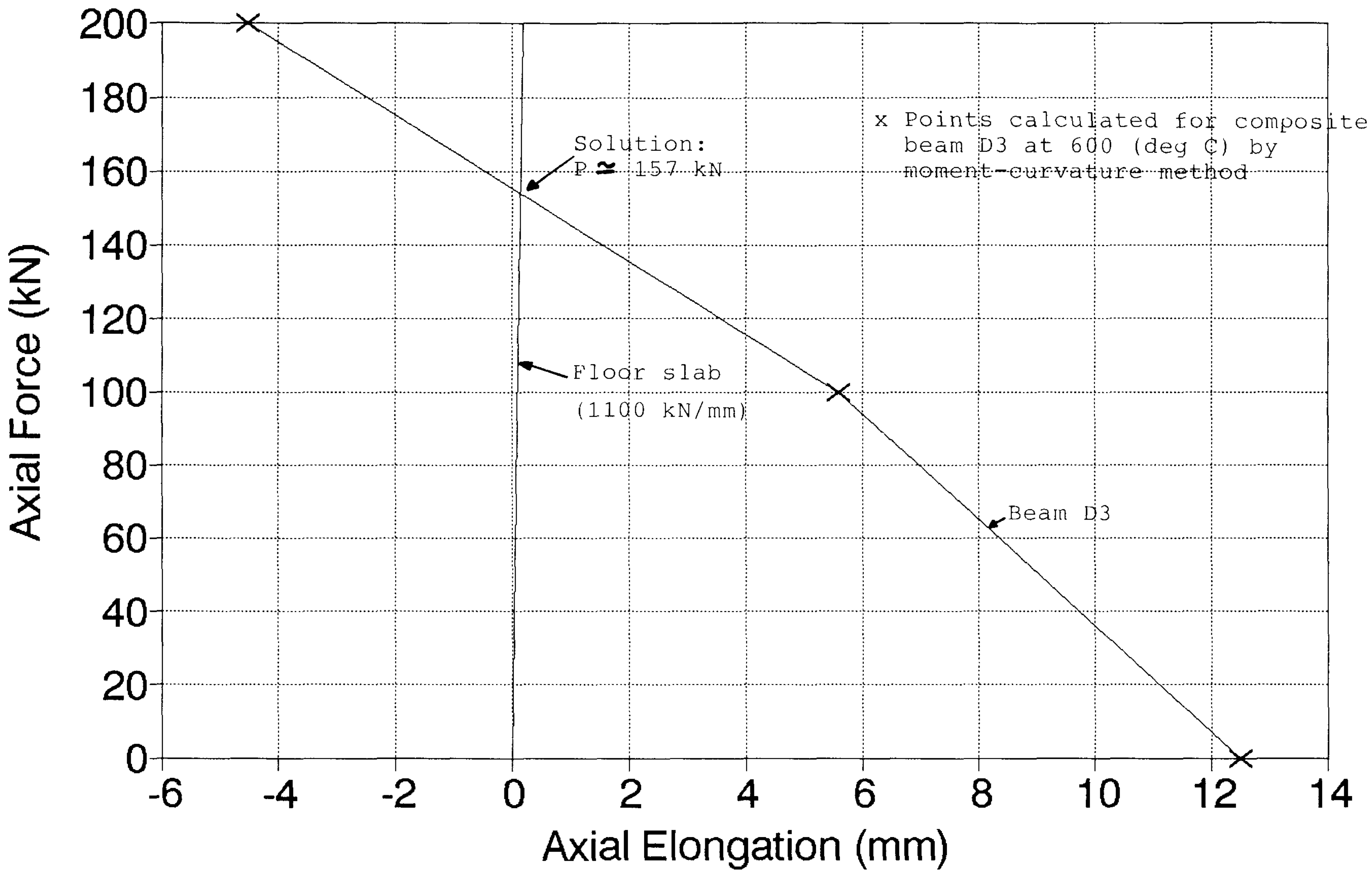


Figure A4: Determination of Axial Force Applied to Beam D3 via Floor Slab

Schedule of Table Numbers:

Table 1 Calculation of Fire Temperatures in the Compartment and at the window for Various Configurations and Fire Load Densities - Ventilation on One Side - Cases A, B and C Ventilation on Two Sides - No Through Draft Assumed - Cases D and E

	Compartment Dimension m			Opening Sizes m				
	Width	Depth	Height	Window	Door	Fire Load kg/m^2	Compartment Fire Temperature ^oC	Temperature ^oC at Window
CASE A	7.2	11.0	2.4	2 x 1.0 x 0.9	Closed	20	524	1240
"	"	"	"	"	"	30	534	1240
"	"	"	"	"	"	40	535	1240
"	"	"	"	2 x 1.0 x 0.9	0.75 x 2	20	740	1061
"	"	"	"	"	"	30	775	1061
"	"	"	"	"	"	40	784	1061
CASE B	5.4	11.0	2.4	1.0 x 0.9	Closed	20	421	1294
"	"	"	"	"	"	30	425	1294
"	"	"	"	"	"	40	426	1294
"	5.4	11.0	2.4	1.0 x 0.9	0.75 x 2	20	721	1031
"	"	"	"	"	"	30	758	1031
"	"	"	"	"	"	40	767	1031
CASE C	3.6	5.5	2.4	1.0 x 0.9	Closed	20	584	1251
"	"	"	"	"	"	30	627	1251
"	"	"	"	"	"	40	631	1251
"	3.6	5.5	2.4	1.0 x 0.9	0.75 x 2	20	842	1076
"	"	"	"	"	"	30	975	1076
"	"	"	"	"	"	40	1041	1076
CASE D	7.2	11.0	2.4	4 x 1.0 x 0.9	0.75 x 2	20	841	1159
"	"	"	"	"	"	30	906	1159
"	"	"	"	"	"	40	927	1159
CASE E	5.4	11.0	2.4	2 x 1.0 x 0.9	0.75 x 2	20	791	1109
"	"	"	"	"	"	30	847	1109
"	"	"	"	"	"	40	927	1109

Note: 1. Fires are ventilation controlled and therefore temperature at window is independent of fire load density.

2. Fire temperatures inside compartment are less than flames at window. Unburnt gases emitted from compartment burn outside the window.

Table 2: Largest Beam and Truss Deflections After Fire (1)

Member	Depth(2) (mm)	Span (m)	Deflection (mm)	Span/ Deflection	Curvature(3) (km^{-1})	Strain(4) (%)
Beams:						
D3	257	6.0	270	22.2	60.0	2.3
E3	257	6.0	259	23.2	57.6	2.2
C3	402	6.0	227	26.4	50.4	2.0
EF15	257	6.0	217	27.6	48.2	1.9
CD20	257	6.0	206	29.1	45.8	1.8
C2	457	6.0	187	32.1	41.6	2.4
AC5	533	13.5	381	35.4	16.7	1.1
EF16	254	6.0	146	41.1	32.4	1.2
CD23	257	6.0	133	45.1	29.6	1.1
EF17	254	6.0	114	52.4	25.3	1.0
AC4	397	9.0	133	67.7	13.1	0.7
Trusses:						
CD5	1000	13.5	552	24.5	24.2	2.7
CD6	1000	13.5	426	31.7	18.7	2.1
CD4	1000	13.5	377	35.8	16.5	1.9
EF5	1000	13.5	128	106	5.6	0.6

Notes:
(1) No deflections available for beams DE12 to DE15 and Trusses DE4 and DE5.
(2) Depth of steel member.
(3) Curvature assumed constant along length, i.e. curvature = 8 x deflection/length^2.
(4) Strain difference over depth of composite member = curvature x (depth + 130).

Table 3: Load Ratios and Limiting Temperatures for Beams

Beam(s)	Applied Moment During Fire (1) (kN.m.)	Moment Capacity at 20°C (2) (kN.m.)	Moment Capacity at 600°C (2) (kN.m.)	Load Ratio During Fire	Temperature from BS 5950 : Part 8 - Table 1 (3)	Table 5 (4)
D3	47.0	313	152	0.112	804°C	828°C
E3	36.8	313	152	0.118	797°C	825°C
EF15	25.0	313	152	0.080	849°C	846°C
C3	79.6	715	352	0.111	806°C	829°C
CD20, CD23	45.3	313	152	0.145	765°C	810°C
C2	150	984	488	0.152	757°C	806°C
AC5	121	1707	844	0.071	874°C	851°C
EF16, EF17	18.8	274	133	0.069	879°C	852°C
AC4	57.6	611	296	0.094	829°C	838°C

Notes:

(1) Loads: Floor slab - 1.77 kN/m^2, Site hut - 1.50 kN/m^2 (grid lines C-D,32-34), Marble floor overlay - 1.75 kN/m^2 (grid lines D-E,32.5-34.5).

(2) Based on average expected material strengths : Steel-400 N/mm^2, Concrete - 40 N/mm^2.

(3) Uniform Temperature by Moment Capacity Method. Based on strength reduction at 2.0% strain.

(4) Bottom flange temperature by Limiting Temperature Method (assumes temperature gradient over beam depth).

Table 4: Load Ratios and Limiting Temperatures for Truss CD4 (Type T2)

Member (Fig. 5)	Tension or Compression	Design Load Ratio (1)	Load Ratio During Fire (2)(3)	Limiting Temperature to BS 5950, Part 8 - Table 1 (5)	Table 5
AB	Tension	0.32	0.16	722°C	722°C
BD	Compression	0.31	0.155	726°C	635°C
DE	Tension	0.30	0.15	731°C	731°C
EG	Compression	0.26	0.13	747°C	635°C
CB	Compression	0.11	0.055	831°C	635°C
FE	Compression	0.11	0.055	831°C	635°C
GH	Compression	0.11	0.055	831°C	635°C
Bottom Chord	Tension	--	0.215(4)	711°C(6)	711°C

Notes:
(1) Load ratios as calculated in fire design document by Smith, Malhotra and Newman.
(2) Loads: Floor slab - 1.77kN/m^2, Site hut - 1.40 kN/m^2, Truss self-weight - 0.9 kN/m.
(3) Obtained by scaling design load ratio (unless noted otherwise).
(4) Calculated as applied moment/moment capacity of composite truss = 244 kNm/1136kNm.
(5) Based on strength reduction at 0.5% strain (unless noted otherwise).
(6) Based on strength reduction at 2.0% strain.

Table 5: Load Ratios and Limiting Temperatures for Columns

Column	Area (mm^2)	Radius of Gyration (mm)	Slenderness[2]	Nominal Yield Stress (N/mm^2)	Strut Compressive Strength (N/mm^2)	Load Capacity (kN)	Applied Load During Fire (kN)[3]	Load Ratio	Limiting Temperature [5]
C32[1]	36600	103	47	340	272.5	9970	1460	0.146	740°C
C33[1]	36600	103	47	340	272.5	9970	1510	0.151	737°C
C34[1]	36600	103	47	340	272.5	9970	1600	0.160	732°C
D32/33/34	80810	110	44	325	251	20300	1500[4]	0.074	779°C
E32/34	29980	102	47	340	272.5	8170	1500[4]	0.184	719°C
E33[1]	22570	95	50	340	265	5980	1500	0.251	682°C

Notes:
(1) Column locally buckled and shortened.
(2) Effective length assumed equal to actual length (4.8m).
(3) Applied loads taken from SOM computations unless noted otherwise.
(4) Assumed value of applied load.
(5) Limiting temperature from BS 5950 : Part 8 : Table 5 (extrapolated values).

Table 6: Structure Stiffness to Produce Column Failure

Column	k_c(kN/mm) (1)	P_u(kN) (2)	Δ P(kN) (3)	ΔL(mm) (4)	k_{eq}(kN/mm) (5)	k_s(kN/mm) (6)	Δ L(mm)	k_{eq}(kN/mm)	k_s(kN/mm)
	$\Delta T = 600^oC$			$\Delta T = 200^oC$			$\Delta T = 400^oC$		
C32	789	4420	2960	13.4	220	305	26.9	110	128
C33	"	"	2910	"	217	299	"	108	125
C34	"	"	2820	"	210	286	"	105	121
D32/33/34	1740	8990	7490	"	557	819	"	279	332
E32/34	647	3620	2120	"	158	209	"	78.9	89.9
E33	487	2650	1150	"	85.6	104	"	42.8	46.9

Notes:

(1) Column stiffness at temperature T (based on CTICM modulus reduction formula).
(2) Load capacity at temperature T (based on BS 5950, Part 8, Table 5).
(3) Additional load to produce failure.
(4) Free thermal expansion of column relative to structure for temperature differential Δ T.
(5) Equivalent stiffness (structure + column) at failure.
(6) Structure stiffness to produce failure (see Appendix 3).

Schedule of Figure Numbers:

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Rosehaugh Stanhope Developments

BROADGATE

MASTER PLAN

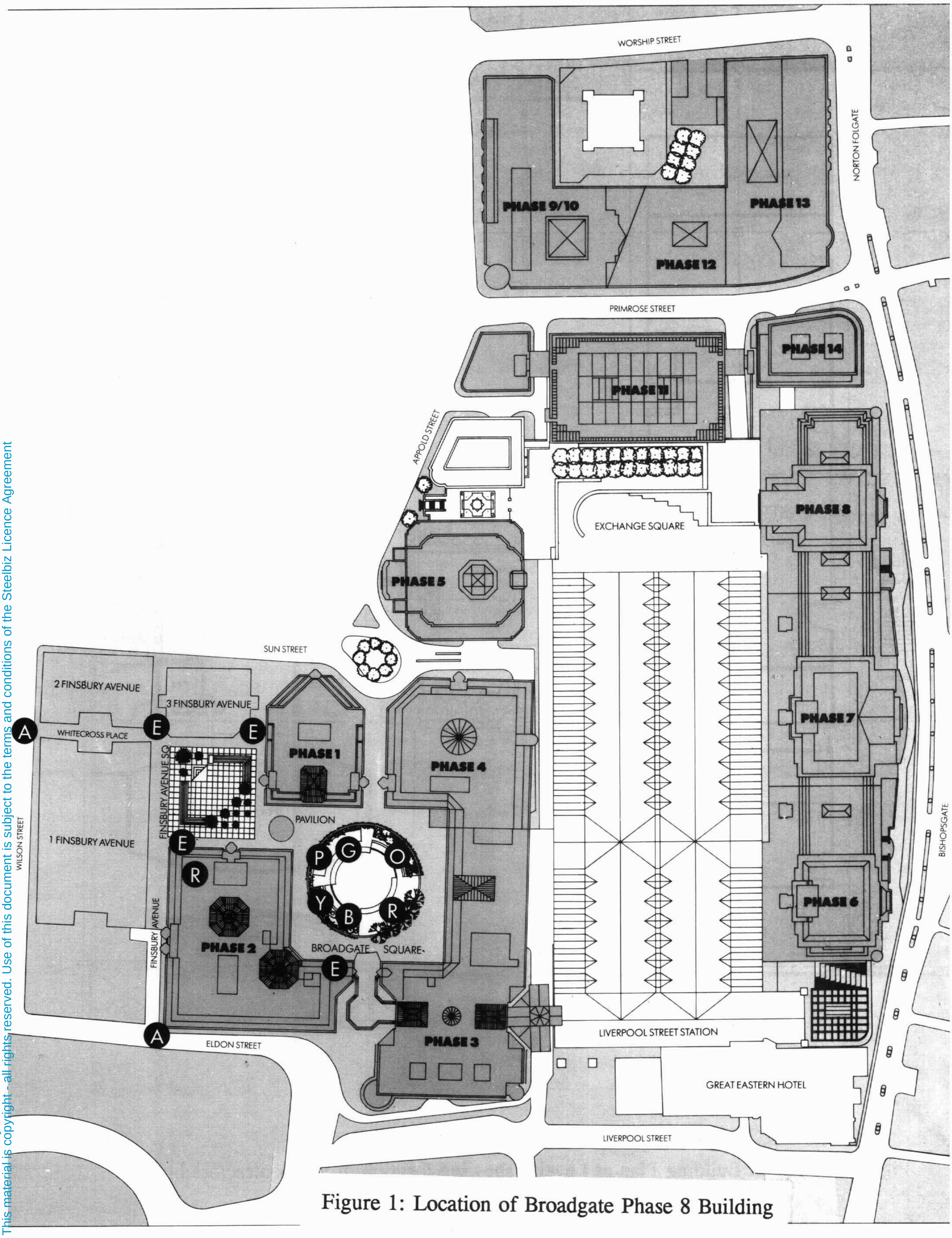


Figure 1: Location of Broadgate Phase 8 Building

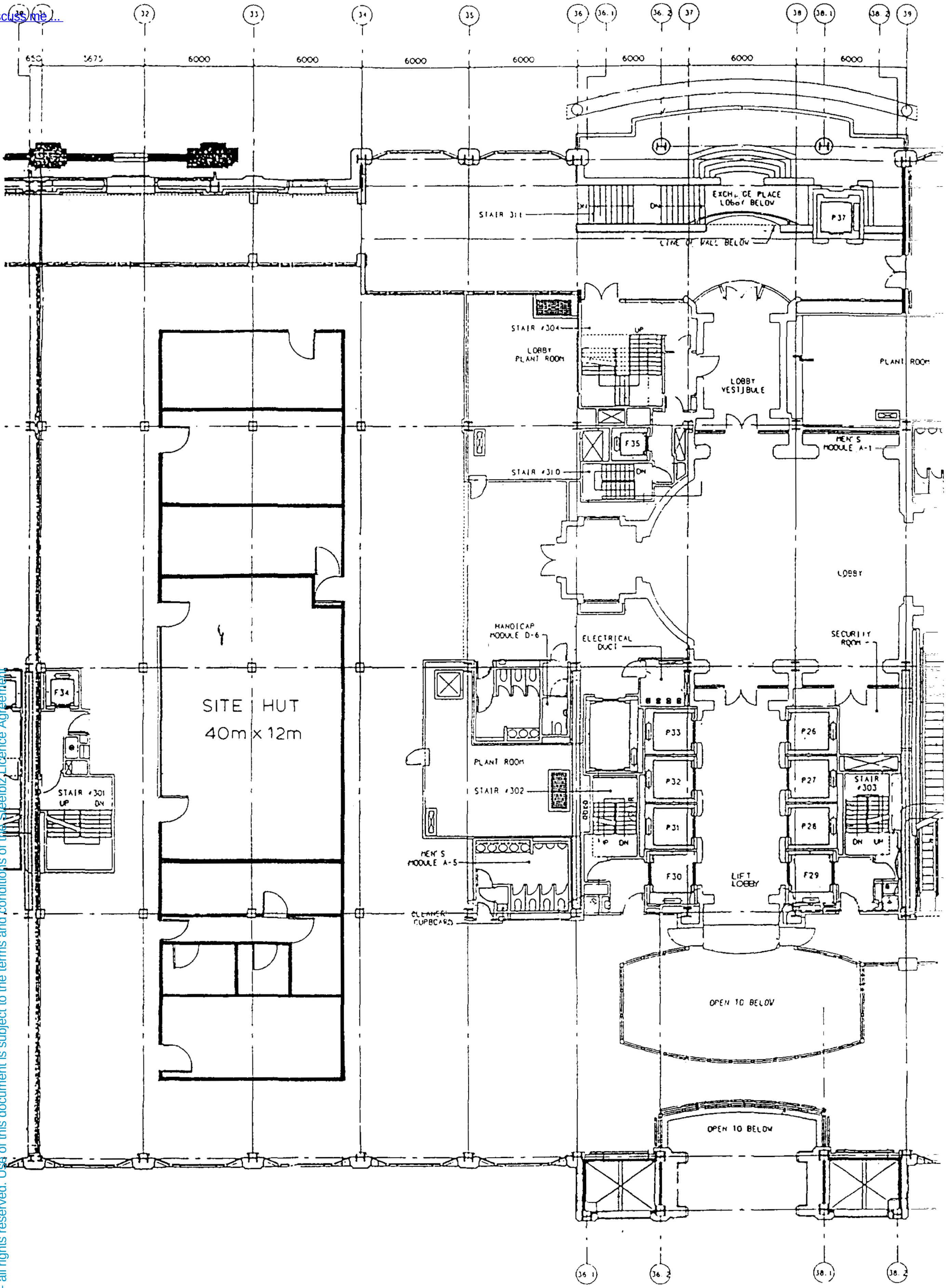


Figure 2: General Building Plan at Level 1 showing location of sub-contractors hut

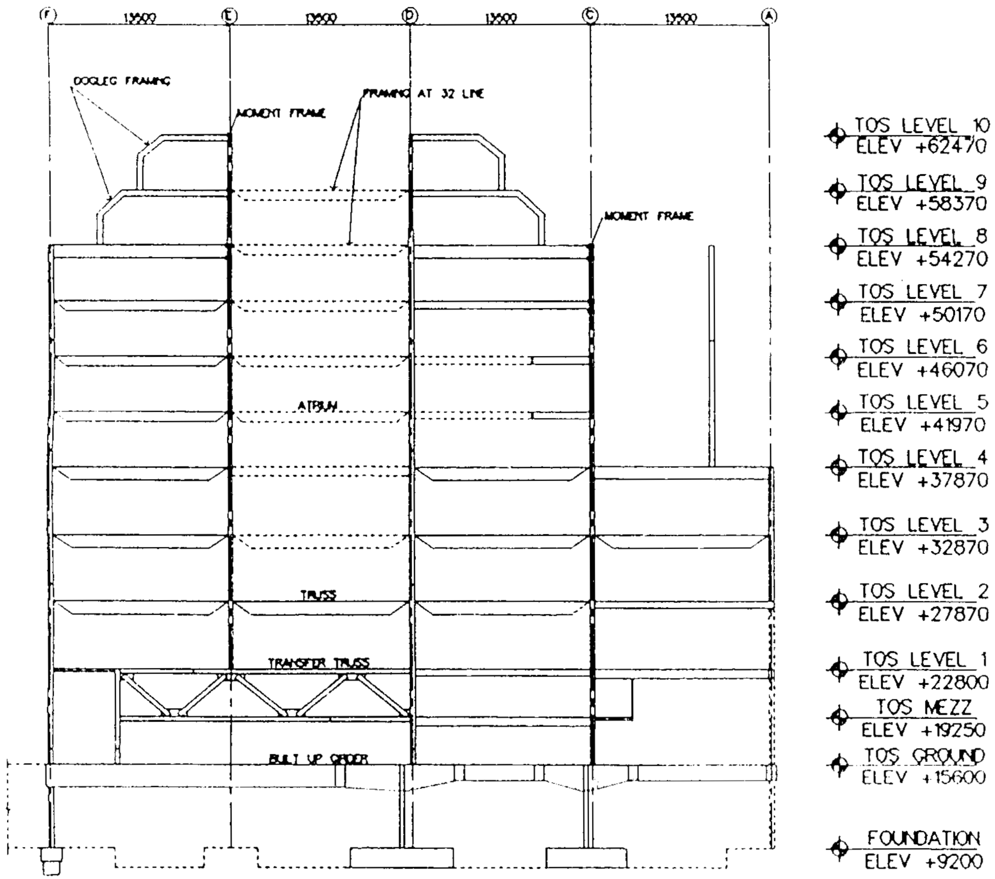


Figure 3 : Sections through the building on grid line 33

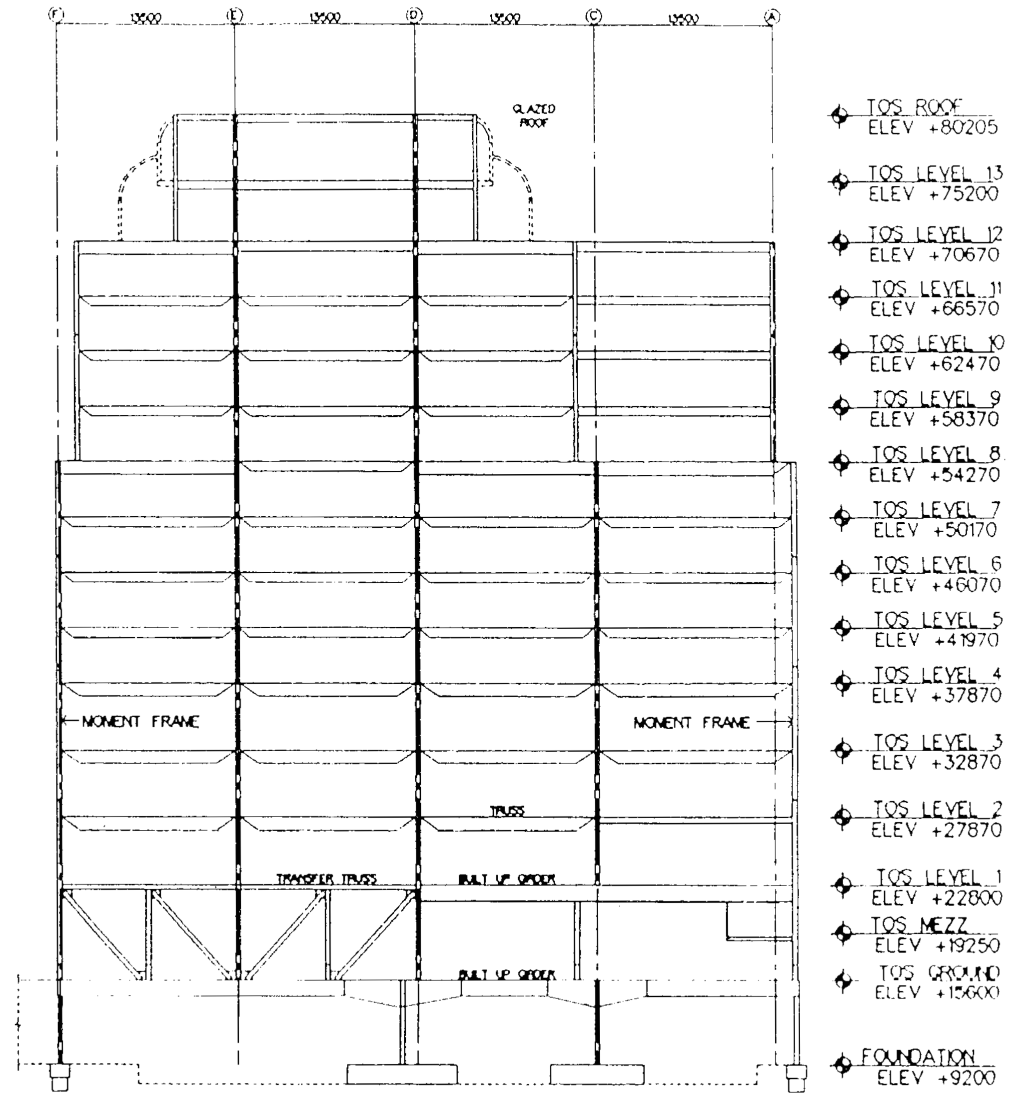


Figure 3 : Sections through the building on grid line 35

A1	TRANSFER GIRDER
AC1	610X229X113
AC2	354X127X33
AC3	610X305X149
AC4	406X140X39
AC5	533X210X92
AC6	356X406X235
AC7	356X406X235
AC8	406X140X46
AC9	BUILT UP GIRDER
AC15	406X140X39
AC16	406X140X39
AC17	406X140X39
AC18	254X102X25
C1	457X152X52
C2	457X152X60
C3	406X140X46
C4	406X140X46
CD1	TRUSS TYPE 2
CD2	TRUSS TYPE 2
CD3	TRUSS TYPE 2
CD4	TRUSS TYPE 2
CD5	TRUSS TYPE 2
CD6	TRUSS GIRDER TG2
CD7	TRUSS GIRDER 2
CD8	TRUSS GIRDER 2
CD20	254X102X25
CD21	254X102X22
CD22	254X102X22
CD23	254X102X25
D2	457X102X25
D3	254X102X25
D4	457X152X67
DE2	610X229X101
DE3	TRUSS TYPE 2
DE4	TRUSS GIRDER 2
DE5	TRUSS GIRDER 2
DE6	TRUSS TYPE 2
DE7	TRUSS TYPE 2
DE12	254X102X25
DE13	254X102X22
DE14	254X102X22
DE15	254X102X25
E1	406X140X39
E2	457X152X67
E3	254X102X25
E4	457X152X67
EF1	TRUSS TYPE 2
EF2	TRUSS TYPE 2
EF3	TRUSS TYPE 2
EF4	TRUSS TYPE 2
EF5	TRUSS GIRDER 2
EF6	TRUSS GIRDER 2
EF7	TRUSS TYPE 2
EF8	TRUSS TYPE 2
EF15	254X102X25
EF16	254X102X22
EF17	254X102X22
EF18	254X102X25
F1	533X210X92
F2	533X210X92
F3	533X210X92
F4	533X210X92

DETAILS OF TRUSSES ON FIG.5

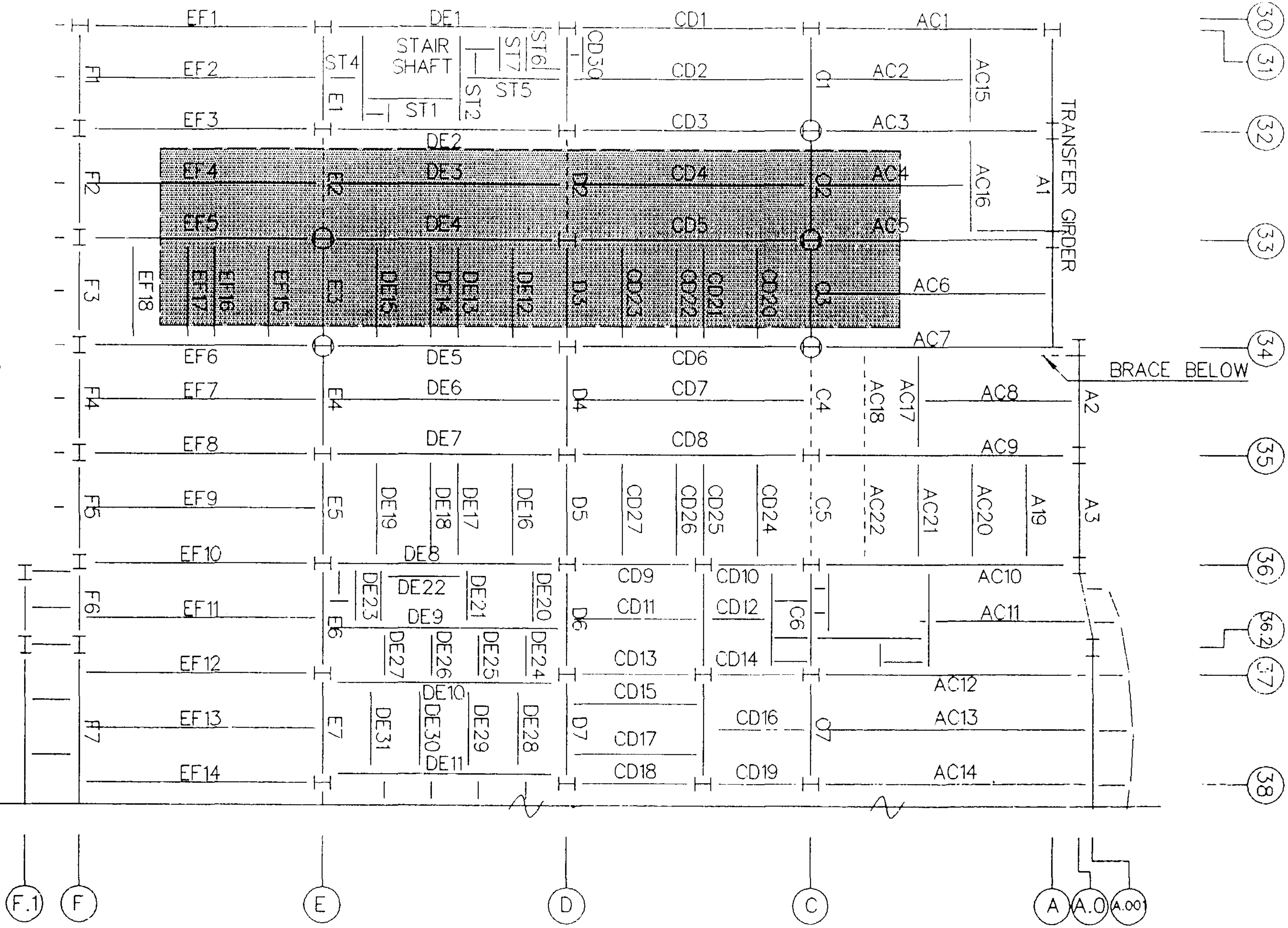
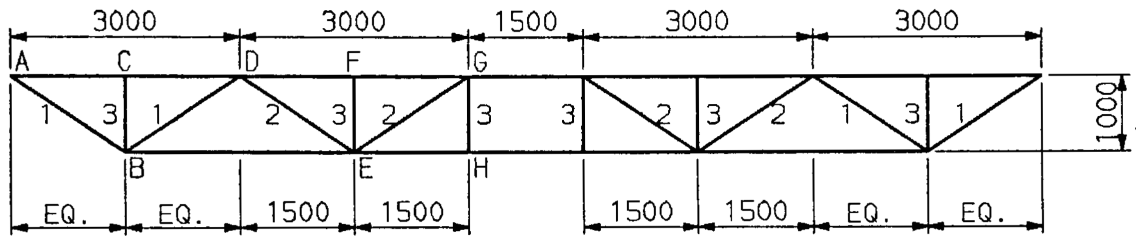


Figure 4: Floor Plan at Level 2 showing members designation

TRUSS T2

TRUSS MARK	DESIGN REACTION (KN)	TOP CHORD	BOTTOM CHORD	WEB MEMBERS (FACTORED FORCE IN KN)			SHEAR STUDS	MIDSPAN CAMBER (MM)
				1	2	3		
T2	300	127x152x19 ST	127x152x21 ST	JL70x70x8 (390)	JL50x50x6 (195)	JL50x50x5 (55)	26	–



TRUSS GIRDER TG2

TRUSS MARK	DESIGN REACTION (KN)	TOP CHORD	BOTTOM CHORD	WEB MEMBERS (FACTORED FORCE IN KN)			SHEAR STUDS	MIDSPAN CAMBER (MM)
				1	2	3		
TG 2	400	305x152x49 ST	305x152x49 ST	JL80x80x10 (600)	JL80x80x8 (250)	JL60x60x8 (210)	56	–

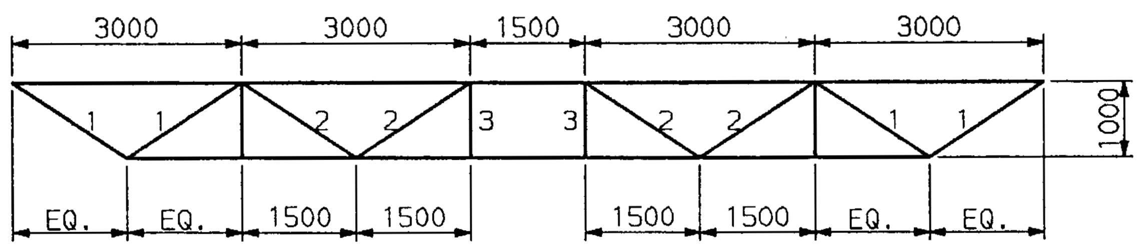


Figure 5: Configuration of Truss T2 and Truss Girder TG2 used at Level 2

Discuss me ...

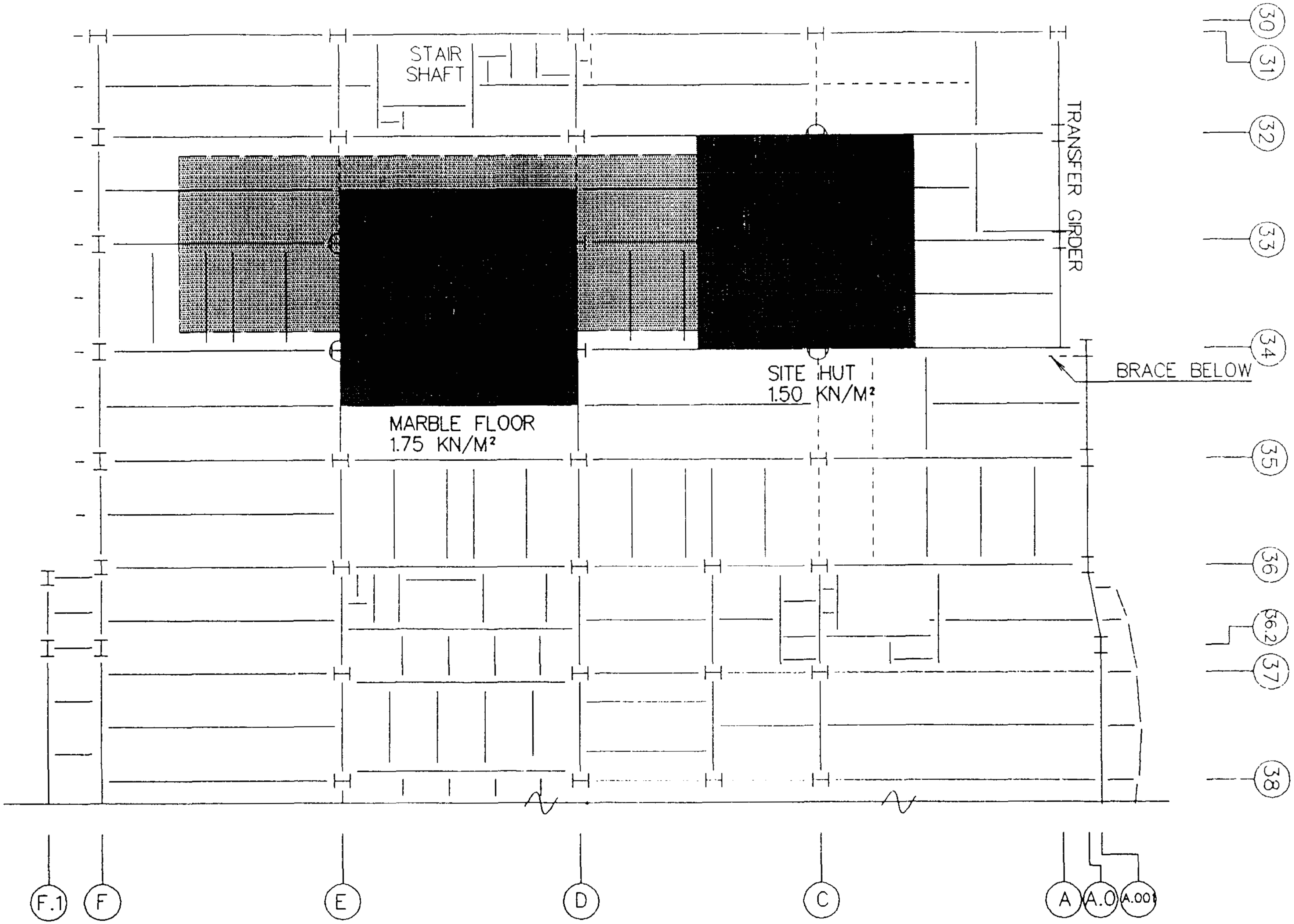


Figure 6: Superimposed Loads at Level 2 During Fire

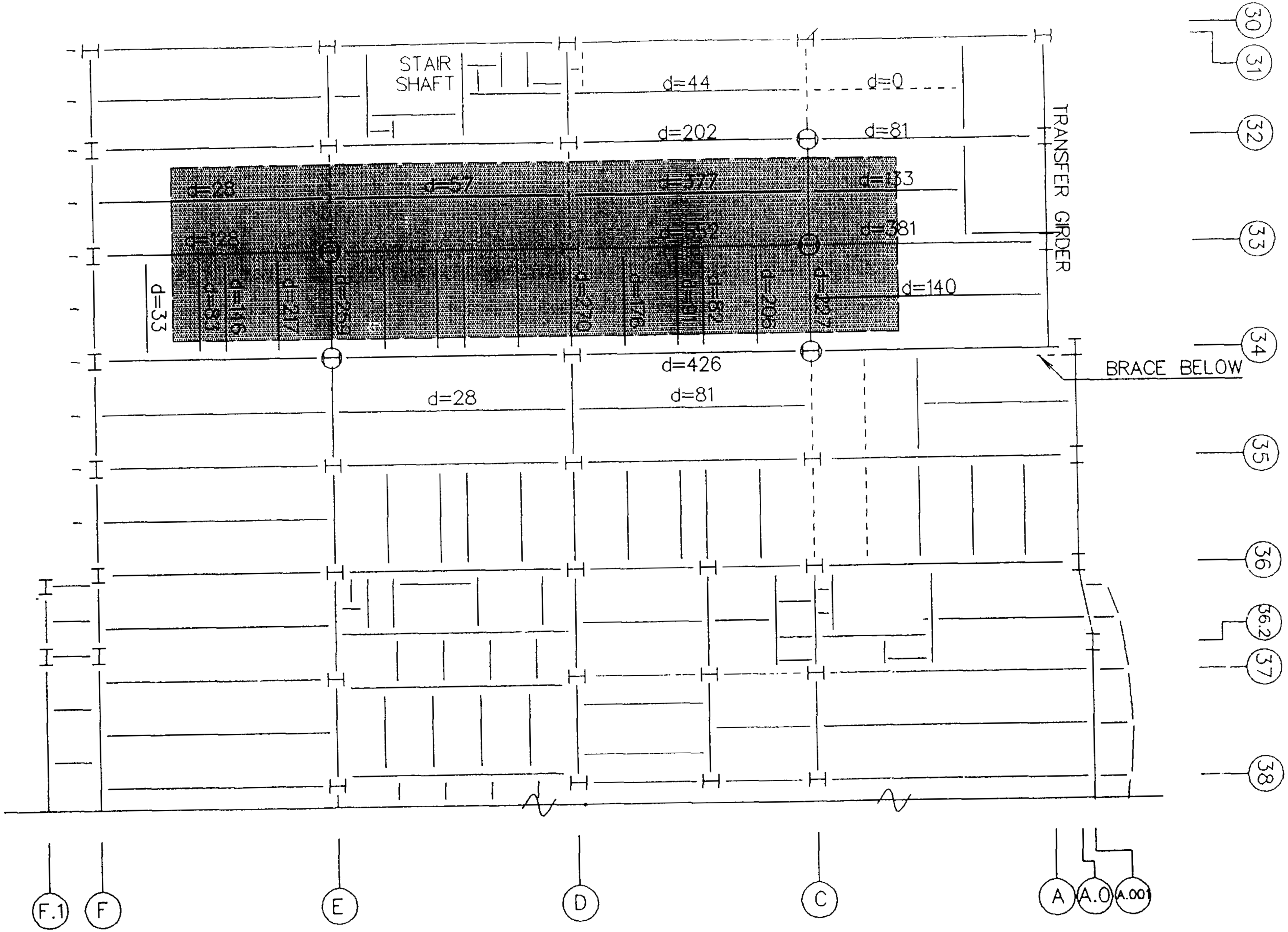


Figure 7: Plan Level 2 showing beam deflection relative to end of that member

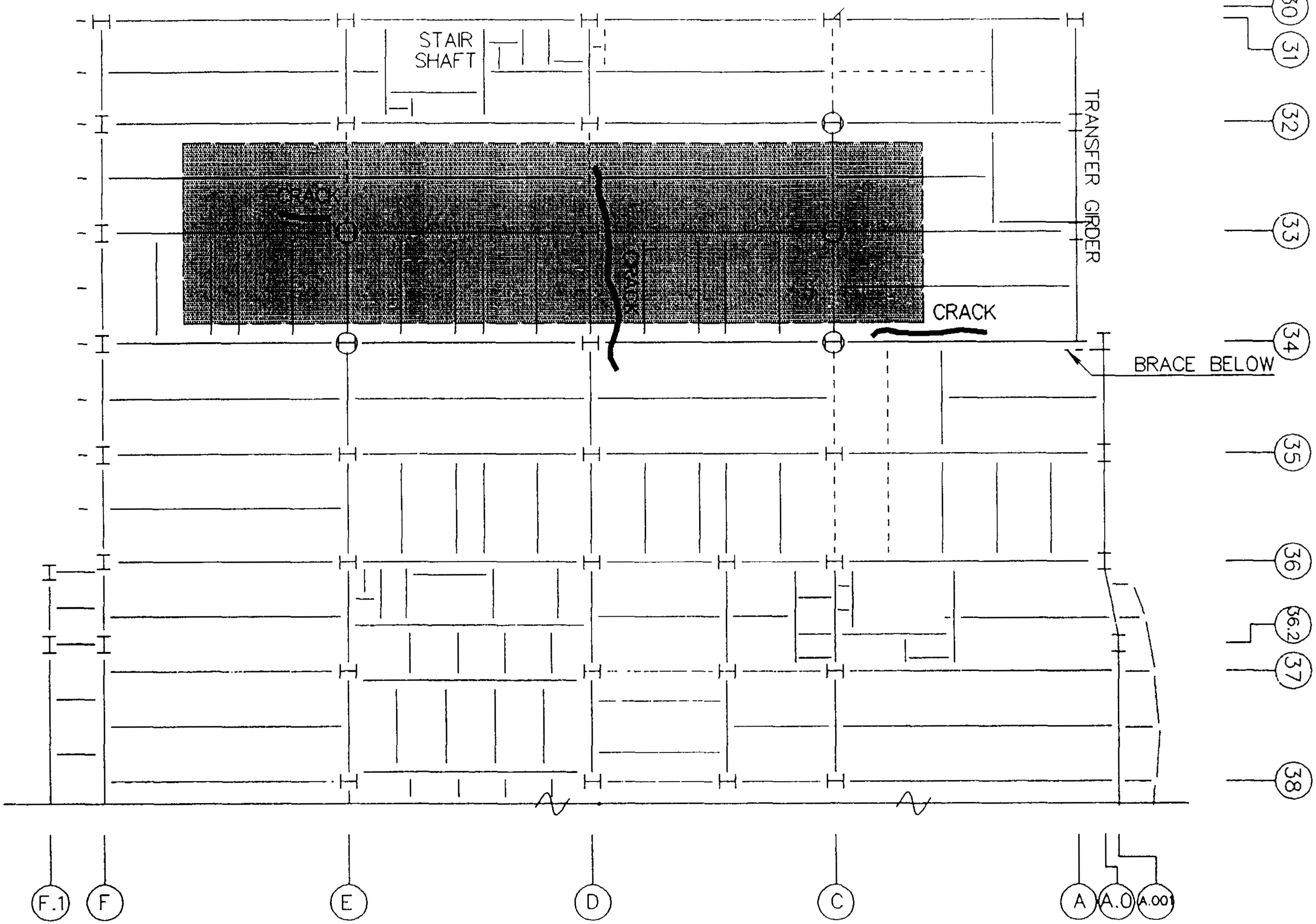


Figure 8: Plan Level 2 showing position of major cracks in concrete floor at Level 2

Discuss me ...

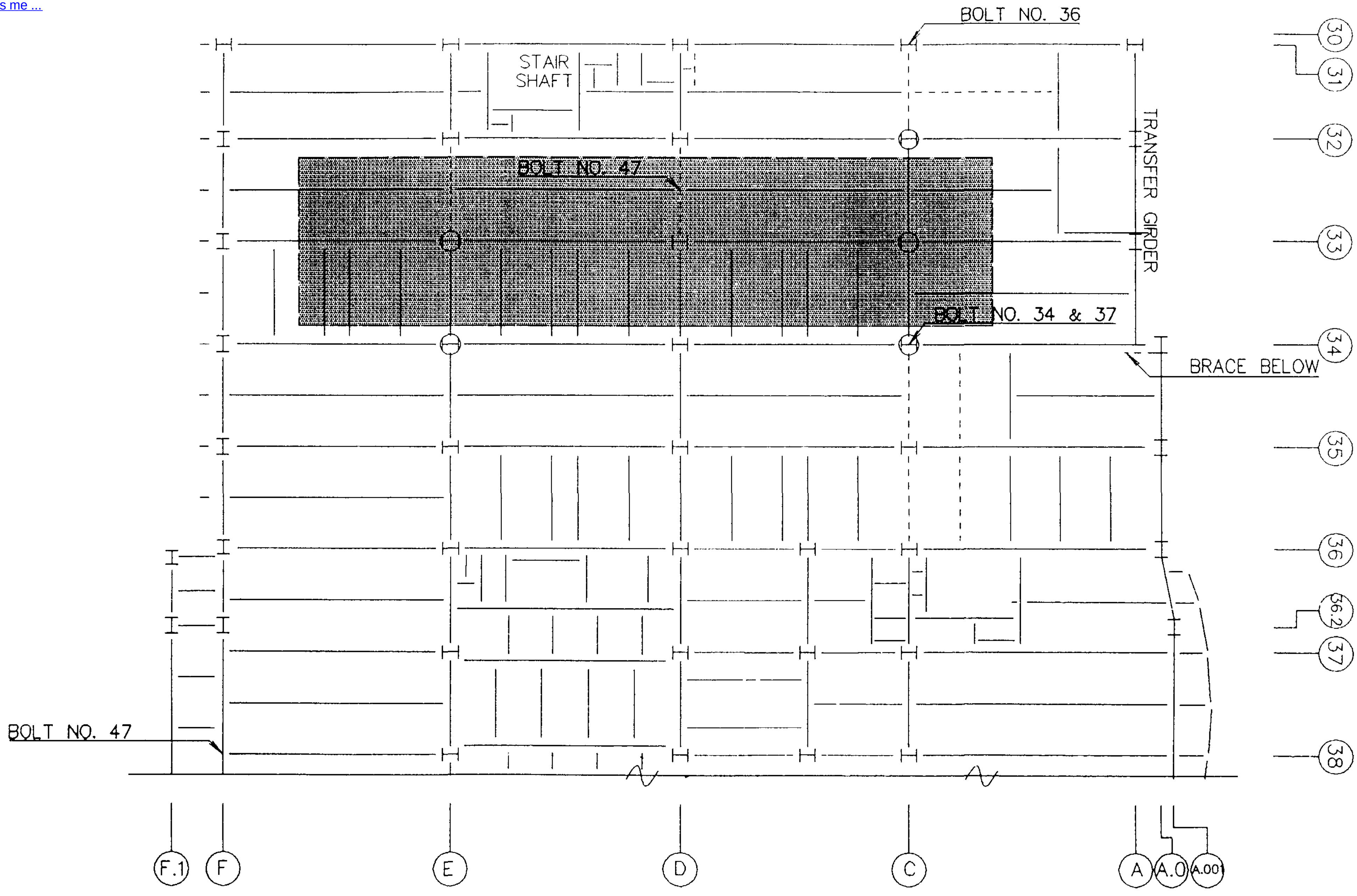


Figure 9: Plan Level 2 showing positions of bolts used in British Steel's investigation

Discuss me ...

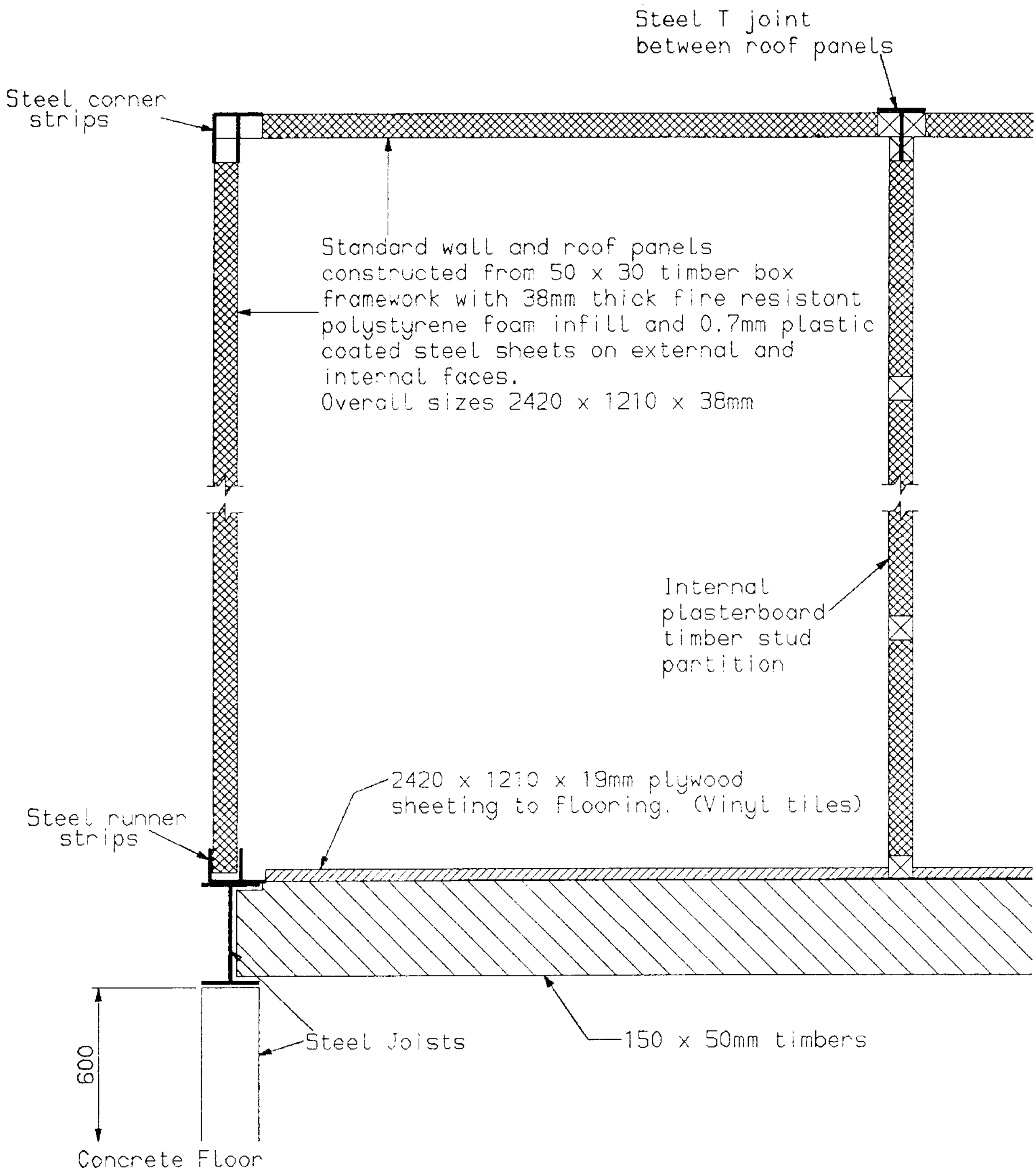
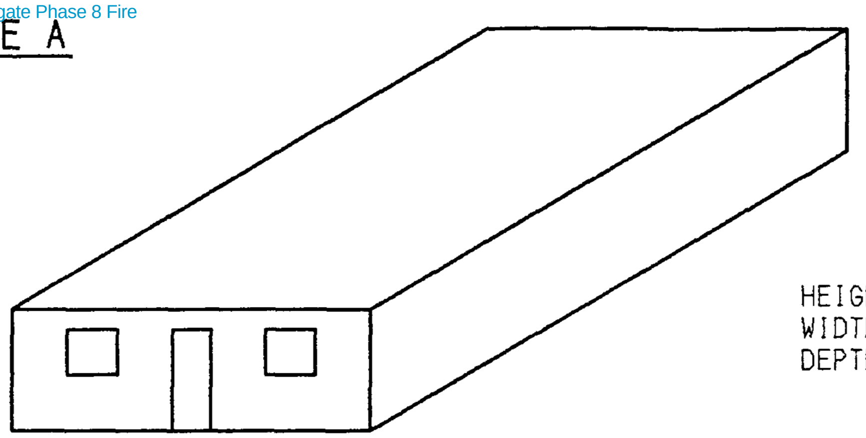


Figure 10: Constructional Details of the Sub Contractors Hut which was destroyed in the fire

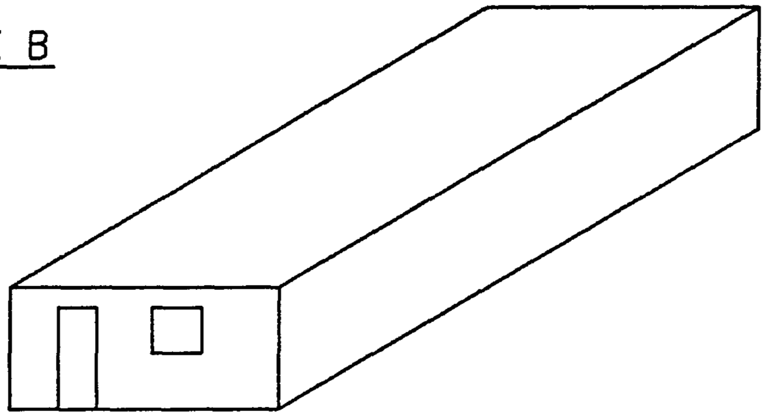
CASE A



HEIGHT = 2.4m
WIDTH = 7.2m
DEPTH = 11m

2 WINDOWS .9m x 1.0m
1 DOOR .75m x 2.0m

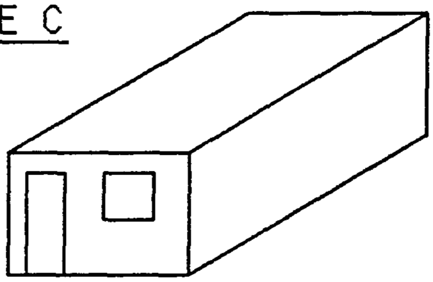
CASE B



HEIGHT = 2.4m
WIDTH = 5.4m
DEPTH = 11m

1 WINDOW .9m x 1.0m
1 DOOR .75m x 2.0m

CASE C



HEIGHT = 2.4m
WIDTH = 3.6m
DEPTH = 5.5m

1 WINDOW .9m x 1.0m
1 DOOR .75m x 2.0m

CASE D has identical room shape to A, but with a door and windows at both ends

CASE E has identical room shape to B, but with a door and window at both ends

Figure 11: Typical Configurations of 'Compartment' within the Sub Contractors hut

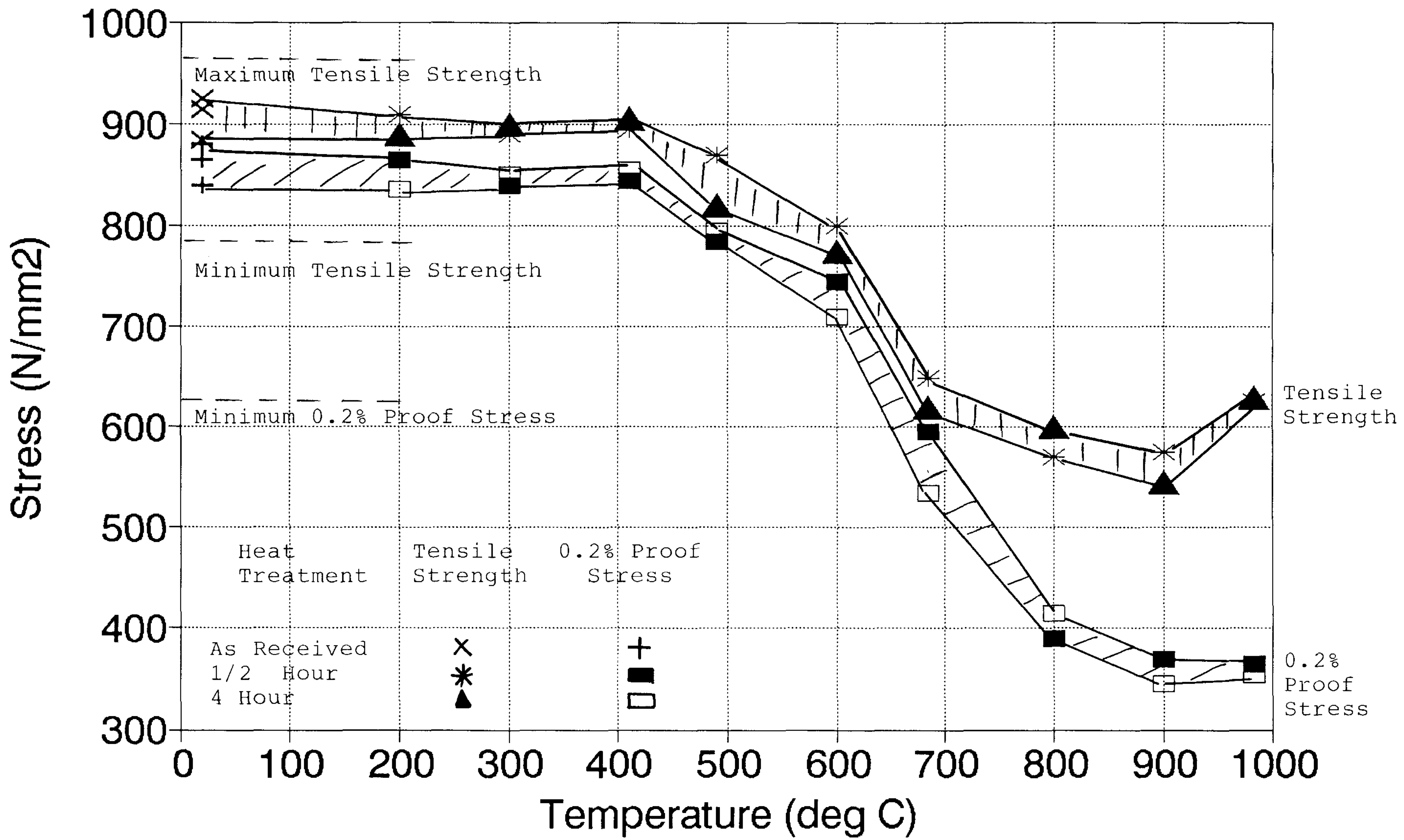


Figure 12: Influence of Fire Simulation Heat Treatment on the Residual Strength of Grade 8.8 Bolts

73

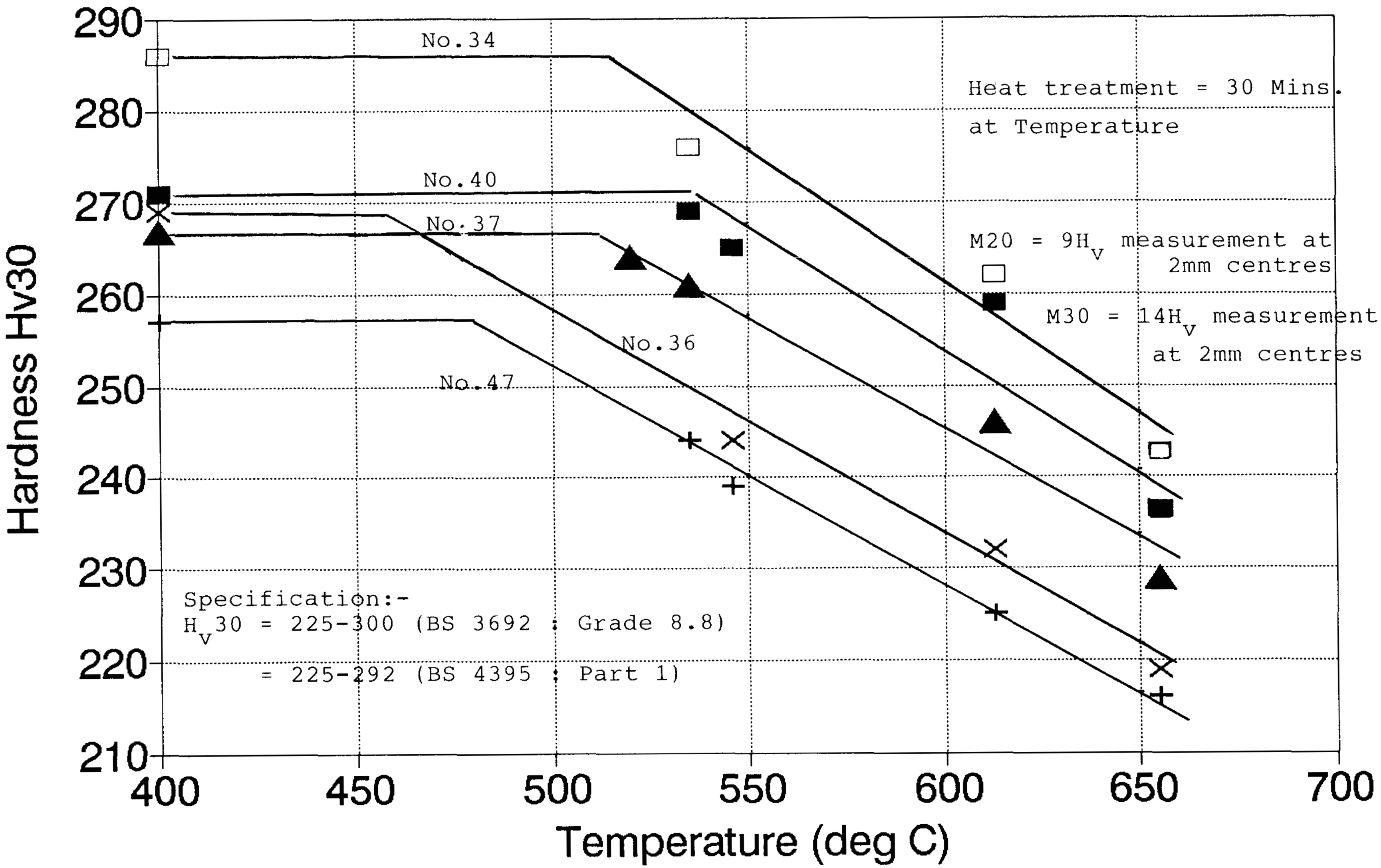


Figure 13: Influence of Tempering Temperature on the Vickers Hardness of Bolts Recovered from the Building

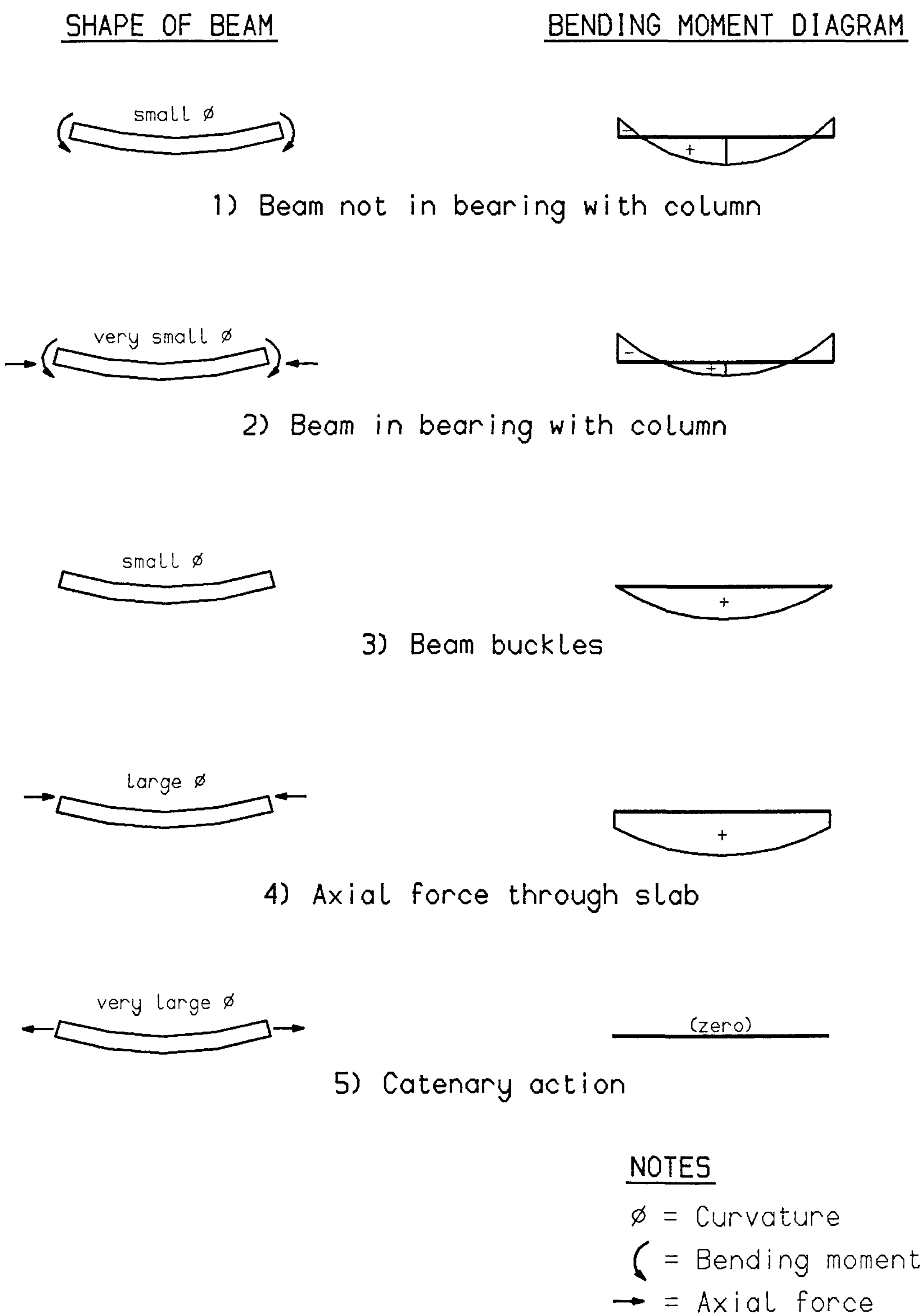


Figure 14: Stages in Behaviour of Restrained Composite Beam

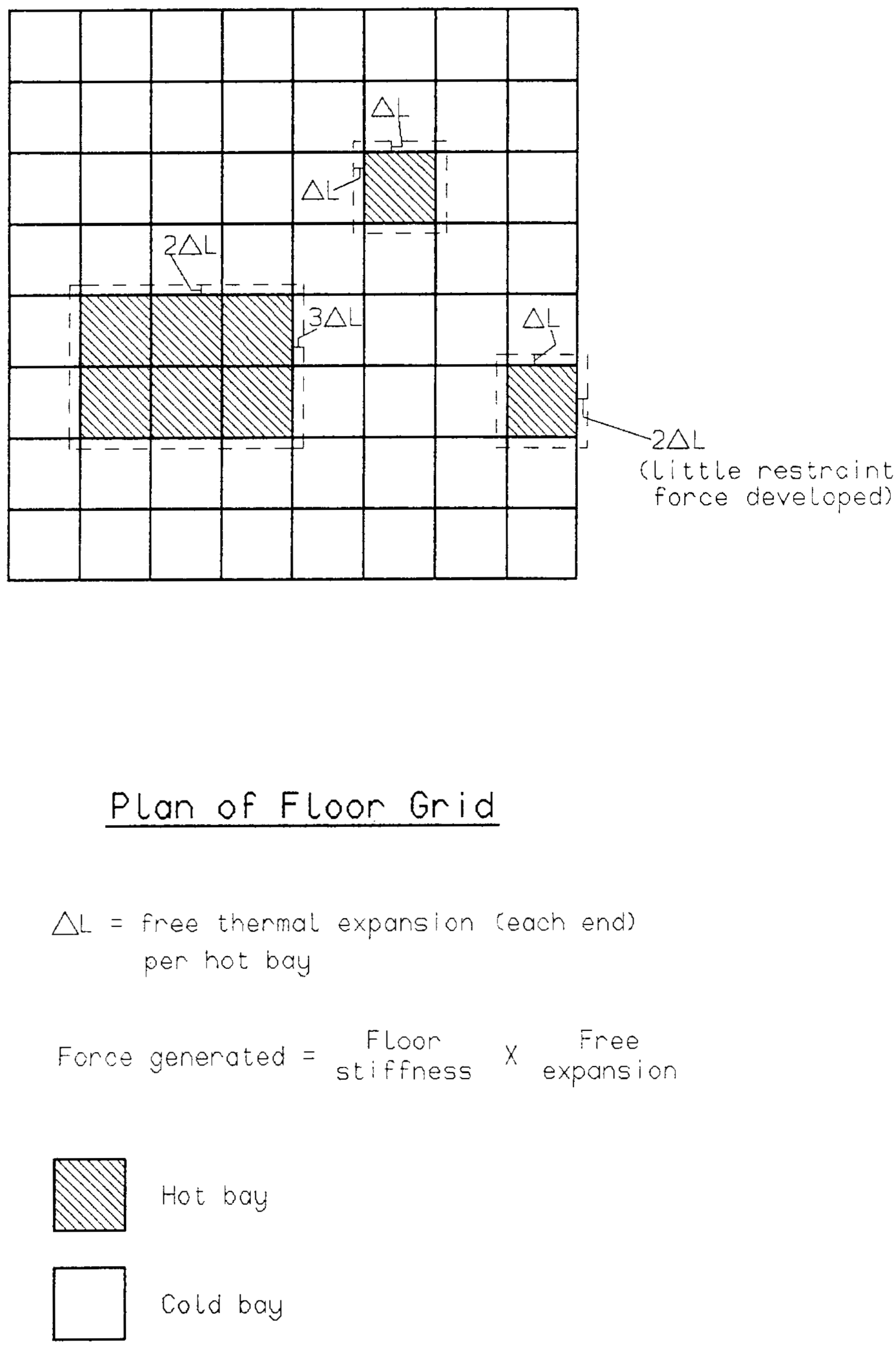


Figure 15: Idealised Representation of Restraint Forces Developed in Floor

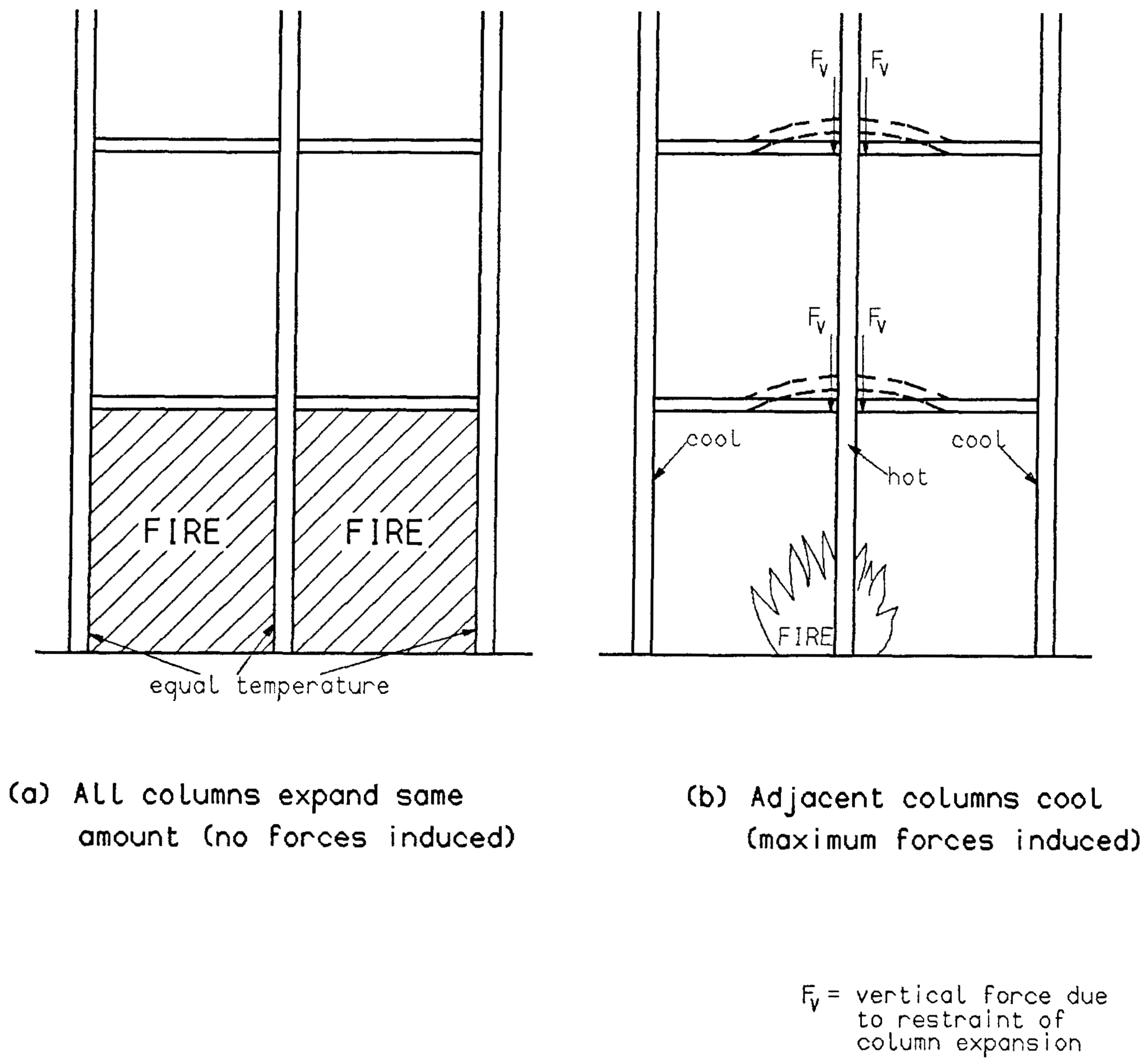


Figure 16: Axial Restraint of Columns

Schedule of Plate Numbers:

Plate 1:	Broadgate Phase 8 from the Exchange Square showing the overall size of the structure taken prior to the fire. No. 12243/4540c. Ian Clook.
Plate 2:	Photograph of a sub-contractor's hut typical of that which was destroyed during the fire. This hut is located at Level Two and was damaged externally during the fire as a consequence of flames through the windows from the fire at the floor below. Note flaking pvc wall coating. LFCDA No. C/19527/AS.
Plate 3:	Fire fighting in progress from Bishopsgate. LFCDA No. C/195271/F.
Plate 4:	Fire damage to the hut. View towards core and Bishopsgate showing columns D32,D33 and D34. Note: a) Numerous deformed sheet steel components from the hut. b) Complete burnout of hut floor in this area. LFCDA No. C/19527/AY.
Plate 5:	Fire damage to the hut. View from Bishopsgate towards core showing columns E33 and E34. Note the charred timber joists and the combustible goods unburnt at this end of the structure. LFCDA No. C/19527/AZ.
Plate 6:	View from Bishopsgate showing columns E32,E33 and E34. Note unburnt portion of hut floor. LFCDA No. C/19527/BA.
Plate 7:	View towards Exchange Square and Phase 7 wall showing damage to the structure. Note: a) The undeformed heavy column D33. b) The deflection of the light beam D3. c) The detachment of the metal deck from the top flange of beam D3. d) The deformation of truss CD5. No. 12315/4540c. Ian Clook.
Plate 8:	View towards Exchange Square and Phase 7 wall showing deformation of the structure. Note: a) The deformed columns C32,C33 and C34. b) The deformed diagonal bracing members of trusses CD5 and CD6. c) The buckling of the lower flanges of beams AC5 and C3 at the column connections. LFCDA No. C/19527/AG.
Plate 9:	View between lines 33 and 34 facing Exchange Square showing deformation of the structure. Note: a) The lack of buckling in beams CD20,CD21 and CD22 supported by the truss compared with the buckling and greater deflection of beam C3 framing into the column. b) The deformation of the metal deck, i.e. crinkling and ballooning out. LFCDA No. C/19527/BC.

Plate 10: View towards Phase 7 and Exchange Square showing column C34.
Note: a) Buckling of column C34.
b) Buckling of lower flange of beam AC5.
c) Uniform deflection of beams AC6 and AC7.
No. 12317/4540c. Ian Clook.

Plate 11: View towards Exchange Square showing column C34.
Note: a) Deformed end of truss CD6.
b) Buckled flanges of beams C3 and C4.
LFCDA No.C/19527/BQ.

Plate 12: View towards Phase 7 of Column E33.
Note: a) Buckling of column.
b) Buckling on lower flange of beam E3.
c) c.d. welded pins on top boom of truss formerly used to fix fire protection in place.
LFCDA No. C/19527/BE.

Plate 1: Broadgate Phase 8 from the Exchange Square showing the overall size of the structure taken prior to the fire.

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Discuss me ...

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Note: a) The undeformed heavy col. D33. b) The deflection of the light beam D3. c) The detachment of the metal deck from the top flange of beam D3. d) The deformation of truss CD5.

Plate 8: View towards Exchange Square and Phase 7 wall showing deformation of the structure.
Note: a) The deformed cols. C32, C33 and C34. b) The deformed diagonal bracing members of trusses CD5 and CD6 c) The buckling of the lower flanges of beams AC5 and C3 at the column connections.

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Plate 9: View between lines 33 and 34 facing Exchange Square showing deformation of the structure.
Note: a) The lack of buckling in beams CD20, CD21 and CD22 supported by the truss compared with the buckling and greater deflection of beam C3 framing into the column.
b) The deformation of the metal deck, i.e. crinkling and ballooning out.

Plate 10: View towards Phase 7 and Exchange Square showing column C34.
Note: a) Buckling of col. C34. b) Buckling of lower flange of beam AC5
c) Uniform deflection of beams AC6 and AC7.

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Plate 11: View towards Exchange Square showing column C34.
Note: a) Deformed end of truss CD6 b) Buckled flanges of beams C3 and C4.

Plate 12: View towards Phase 7 of Column E33
Note: a) Buckling of column b) Buckling on lower flange of beam E3
c) c.d. welded pins on top boom of truss formerly used to fix fire protection in place.