

Experience of Fires in Concrete Structures

by A. K. Tovey and R. N. Crook

Synopsis: This paper outlines the procedures adopted in obtaining information on fire-damaged concrete structures since 1975. Details are given on the information received from questionnaires and a summary of the building and construction types, damage and repairs are tabulated. The general conclusion is that concrete structures behave well under fire conditions with the majority of cases being repairable.

Keywords: buildings; concrete construction; damage; fire resistance; fires; performance; prestressed concrete; reinforced concrete; surveys

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INTRODUCTION

A small number of major fires in concrete structures have been well investigated and reported in various publications (1-5). These indicate the good fire resistance performance of both reinforced and prestressed concrete structures but, in order to gain a more comprehensive picture, the Fire Resistance Committee of The Concrete Society, England, decided to investigate a large number and variety of fire-damaged concrete structures within the UK.

An initial questionnaire, as given in Appendix A, was prepared and distributed to several hundred UK consultants in order to obtain contact with as many people as possible having experience of fire-damaged concrete structures.

Having obtained the contact, the next stage was to obtain detailed information on the structures concerned and, in order to be consistent, a comprehensive record summary sheet as given in Appendix B was developed. This is also to be issued as a questionnaire in future cases of fires.

RECORDING OF DATA

The record summary sheet is divided into six sections. The first part is concerned with the identification of the building and the contact point for information. Section 2 requires a description of the building, particularly its design and construction, in order to have a clear idea of the type of structure before the fire and its potential resistance to fire. Of particular interest is the question of structural continuity. If this was present, it could have had a significant influence on the structure's behaviour pattern and details or an estimate of this are requested. Section 3 aims to establish the type of fire

that occurred. It may be that either the fire brigade or subsequent examination by an expert established the severity of the fire. Such information provides a base against which the actual behaviour of the construction can be judged. Section 4 asks primarily for the visual record of damage by spalling and its extent. Spalling occurs in most fires but it is necessary to establish the circumstances under which it may have serious consequences. Section 5 asks for data on structural damage to the parts affected by the fire and how the extent of the damage was established. This is followed by information in Section 6 on the repair work that had to be undertaken to put the building back into commission. A question is asked in the case of reconstruction as to whether the decision was based on the nature of the damage or because of other practical reasons.

BUILDINGS EXAMINED

As a result of this investigation, detailed information has been gathered on the performance, assessment and repair of over 100 fire-damaged concrete structures. The information was obtained from consultants and from the Cement and Concrete Association (England) whose staff have visited and advised on the reinstatement of many fires in concrete structures within the UK. The types of building examined, the type of construction together with the degree of damage and decision in respect to repair is summarized in Appendix C. From this, it will be seen that the structures covered include dwellings, offices, warehouses, factories and car parks of both single and multi-storey construction. Some fires under bridge structures were examined but only one is listed in Appendix C. The forms of construction examined included flat, trough and waffle floors and associated beams and columns of in situ and precast construction in both reinforced and prestressed concrete.

Examining the items under damage and repair shows that:

- (1) most of the structures were repaired and many of those which were not could have been but were demolished for reasons other than the damage sustained;
- (2) much spalling occurred but most structures still performed satisfactorily even those where supplementary reinforcement might now be required under current UK provisions;
- (3) almost without exception the structures performed well during and after the fire.

It is hoped that these investigations will provide essential information on the behaviour of concrete structures in fire to enable recommendations given in the Codes and other relevant documents to be updated. The intention is to prepare a publication giving detailed findings of the investigation. The

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information gathered contains details of tests on components and methods used for repair. To date, the reports show the performance of concrete structures in fire is generally excellent.

An example of the information that is being obtained may be illustrated by reference to Figure 1. This shows an example of damage to the main structural beams to an in situ reinforced concrete warehouse built around 1930. The fire occurred in 1983 and there was noticeable separation of the floor slab and buckling of the main reinforcing bars over the central section of the beam. The omission of stirrups over the central section has clearly demonstrated a detailing deficiency. Such details would not generally be permitted under our current UK Codes but, despite this, the structure adequately withstood the fire. It is clear that examination of such cases can identify structural deficiencies and hence, when appropriate, be used to modify the Code provisions.

Another case which illustrates a detailing deficiency is shown in Figure 2. In this case, the structure consisted of a continuous reinforced concrete waffle floor slab that was detailed to cater for the appropriate negative and positive bending moments along the span. In accordance with the bending moment envelope, the bottom reinforcement can be stopped short of the spans. This is adequate for strength conditions but the effect of the fire caused the unsupported bottom bars to buckle away from the support zone. The Joint Report (1) recommends that 50% of the bottom reinforcement should continue to the support and that 20% of the top bars should be continued across the span. Again, the structure performed adequately in the fire but had all reinforcement be taken to the support then the element would ultimately have performed better and there would probably have been less spalling and damage.

The building fires investigated varied from minor fires of short duration through to major fires reaching, and at times exceeding, the designed fire period. The equivalent fire period was determined, where possible, from examination of debris, tests on the elements and consideration of the fire load.

The evidence to date shows all structures to have behaved well, even those having less than satisfactory details. Improving detailing can significantly improve the performance under fire and thus, if these are incorporated into the Codes, it should be possible to relax some of the Code provisions in respect to notional fire resistance requirements and yet still maintain a sufficient factor of safety in the event of a fire.

INVESTIGATION OF BEHAVIOUR OF STRUCTURE

The working group responsible for the review is seeking to obtain data on the overall performance of the structures and, in particular, on items which have prompted some concern in the past.



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One such area being investigated is the overall effect of expansion and contraction on the complete structure. Although expansion of a hot suspended floor of a building can cause bending and shear to be developed in supporting columns and other members, significant problems arising from such are indeed rare within the UK. In fact, the data already collected has not yielded a single instance where significant damage has been noted in a concrete frame remote from the fire. The only collapses recorded were those resulting from instability of other structural materials.

Another area of concern in the UK is the loss of strength or restraint on cooling which according to the Joint Report (1) "could lead to rapid collapse". Some loss of strength of concrete on cooling is not in dispute but the essential point is that the beneficial restraint due to the effects of expansion obtained when hot is additional to the strength normally expected during a fire. If such benefit is lost on cooling then, according to the severity of the fire, the member still has the appropriate proportion of strength due to its normal methods of carrying load at normal temperature. Once again, no immediate evidence has been obtained which shows adverse or unacceptable behaviour on cooling. Nevertheless, the data is to be re-examined to identify those fires where restraint may have had a major influence on the structural behaviour.

The evidence so far indicates that the major reserve in strength comes from continuity reinforcement at supports rather than from expansive restraint during the fire. The former has the benefit in that the strength is also retained on cooling.

ACKNOWLEDGEMENT

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APPENDIX A

INITIAL QUESTIONNAIRE

1. Have you had experience of a fire-damaged structure since 1975? Yes/No
2. What type of structure was involved:
 - a) Shed type
 - b) Multi-storey
 - c) Other
3. What was the type of construction:
 - a) Reinforced concrete
 - b) Precast concrete
 - c) Other
4. What type of concrete was damaged:
 - a) Dense
 - b) Lightweight
5. How much of the structure was damaged:
 - a) All
 - b) Part
6. Was the structure:
 - a) Demolished
 - b) Repaired
7. Please give the name and address of a contact for further details:
Name
Firm
Address
.....
.....
Tel.No.
8. Are there any other comments you wish to add.



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APPENDIX B

FIRES IN CONCRETE STRUCTURES

Summary Record No.....

1. REFERENCE DATA

- 1.1 Name of the building:
- 1.2 Address:
- 1.3 Contact:
- 1.4 Firm:
- 1.5 Telephone No:

2. BUILDING DESCRIPTION

- 2.1 Type of occupancy:
- 2.2 Contents:
- 2.3 Plan size:
- 2.4 Height / No. of storeys
- 2.5 Type of construction (reinforced, prestressed, in situ, etc):
- 2.6 Brief description:
- 2.7 Aggregate type:
- 2.8 Cover to reinforcement (design / actual):
- 2.9 Supplementary requirement:
- 2.10 Detailing for continuity:
- 2.11 Structural design by and construction date:
- 2.12 Fire resistance requirement:

3. FIRE DETAILS

- 3.1 Date of fire:
- 3.2 Fire duration:
- 3.3 Fire brigade:
- 3.4 Estimation of fire severity (e.g. BS 476 equivalence):
- 3.5 Examination of debris for fire damage assessment:

4. FIRE EFFECTS

- 4.1 Extent of fire spread (No. of storeys, floor areas involved):
- 4.2 Extent of structure affected:
- 4.3 Extent of spalling (localized or extensive):
- 4.4 Reinforcement exposed (locally or extensively):
- 4.5 Spalling damage to
 - floors:
 - beams:
 - columns:
 - walls:

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5. STRUCTURAL DAMAGE ASSESSMENT

5.1 Name of assessor:

5.2 Brief description of damage floors:
 beams:
 columns:
 walls:

5.3 Method of damage assessment:

5.4 Residual deformation:

6. REPAIR

6.1 Was the damaged part repaired or replaced?

6.2 Method of repair:

6.3 If replaced, the reason:

7. GENERAL

7.1 Has there been a previous fire in the building?

7.2 Is a report or photographs of the incident available?

7.3 Can all or some of the information given be published?

APPENDIX C

CONCRETE SOCIETY - FIRE RESISTANCE COMMITTEE

Working Group ASUMMARY OF INFORMATION ON
FIRES IN CONCRETE STRUCTURES

Ref No.	Type of Building	Type of Construction	Damage	Repaired ?
1	Office block	In situ hollow pot	Localized spalling	Yes - reconcreted
2	Supermarket	Prestressed double tee beams. In situ frame	Spalling of beams	Yes - gunite/ reconcreted
3	Office block	In situ waffle	Spalling to ground floor soffit	Yes - gunite
4	Office block	In situ RC	Spalling to soffit one floor	Yes - reconcreted
5	Information not received *			
6	Department Store	In situ RC	Spalling to soffit one floor	Yes - bonded mortar
7	Office block	In situ RC		Yes
8	College	Precast Prestressed	Localized to service shaft	Yes steel sub-frame
9	Bus Garage	Steel frame + concrete	Severe	No - demolished
10	Office block	Steel frame + concrete	Spalling of beams	Yes - reconcreted
11	Furniture shop	In situ RC	Spalling to ground and first floor	Load test Yes - epoxy mortar
21	Multi-storey	RC and PS	Local damage	Yes
22	Office block	Steel frame + concrete	Spalling of beams	Yes - gunite
23	Warehouse	In situ + prestressed pot floor	Spalling of beams	No - demolished
24	Office block	In situ RC frame hollow pot floor	5 floors damaged	Yes/no Part demolished repair by gunite

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APPENDIX C (continued)

Ref No.	Type of Building	Type of Construction	Damage	Repaired ?
25	Warehouse	Precast portal	Damage to single unit	No - demolished Replaced with PC units
26	Hospital	In situ frame	Basement	Yes - gunite
27	Tunnel	In situ RC	Spalling to entire soffit	Yes - major gunite - minor epoxy
28				
29	Office block	In situ	Spalling to soffit 25 mm hard soot	Yes/no demolished bad areas
41	Library	In situ	Limited	Not required
42	Office block	In situ	Basement soffit	Yes/no some demolished gunited areas columns jacketed
44	Power Station	Steel frame + concrete	Thermal expansion Distortion of steelwork Spalling to concrete	Yes
45	Oil jetty	Concrete on steel piles	Severe damage to superstructure Deck burnt through (hydrocarbon fire)	No - demolished and replace superstructure
46	Office block	In situ	Spalling of beams	Yes - bonded mortar
47	-	Steel frame	-	-
48	Office block	In situ	Whole building to some degree Fire burnt for one week	Yes/no some members replaced others repaired
61	Warehouse	In situ	Considerable	-
62	School	In situ + Precast	First floor soffit	Yes structural steel stiffeners
63	Market	Steel frame + precast/prestressed units	Steel - bad damage conc - minimal	Yes/no steel replaced concrete - gunited

APPENDIX C (continued)

Ref No.	Type of Building	Type of Construction	Damage	Repaired ?
64	Foundry	In situ	100 mm spalling to basement floor (molten steel)	Yes - gunite
65	Power Station	RC and steel frame PC roof	-	Yes -
66	Portal frame	RC	-	Yes - epoxy mortar
67	Hospital	In situ trough	Spalling to soffit	No - damaged part demolished suspect original conc.
81	Hotel	Masonry + precast	Spalling to soffit Cracking of masonry walls	Yes/no part demolished mainly repaired
82	Multi-storey	In situ		Yes
83	Multi-storey	In situ		Yes
84	Multi-storey	In situ + precast		Yes
85	Information not received *			
86	Factory			Yes
87	Various	In situ, precast and steel		Yes
88	Multi-storey	In situ		Yes
89	Multi-storey	In situ		Yes,
90	Multi-storey	In situ		Yes
101	Workshop	Precast portal	Shearing of scarf joints	Yes - steel brackets added
102	Information not received *			
103	School	In situ, precast prestressed beams	Expansion of steel bracing punched through concrete box sections	Yes
104	School	Precast panels prestressed purlins	Damage to purlins	Yes - external plates glued to purlins
105	Information not received			

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APPENDIX C (continued)

Ref No.	Type of Building	Type of Construction	Damage	Repaired ?
106	Boiler House	Precast frame, omnia floors, aac roof slab	Damage to roof beams	Yes
107	School	In situ + timber roof	Roof collapsed	No - demolished although cols sound
108	Information not received *			
109	School	In situ walls	Steel roof collapsed. Limited spalling	Yes
110	Information not received *			
E1	Shop	Steel beams + precast slabs	Spalling of plaster + two slabs	Replastered Replaced two units
E2	College	In situ + Precast	General spalling	Yes - gunite
E3	Factory	In situ + Precast	Slight spalling	No - precast beam replaced
E4	Sports Centre	Steel frame + in situ floor	Frame collapsed some spalling to concrete	Yes/no - frame demolished concrete repaired
C1	Basement car Park to high rise flats	In situ frame waffle slab	Spalling to soffit	Yes - bonded mortar
C2	Flats	In situ + Prestressed	Spalling to soffit	Yes
C3	Bridge	In situ	Spalling to beam	Yes - bonded mortar
C4	Flats	In situ		Yes
C5	Flats	Timber frame on concrete raft	Structural frame destroyed. Spalling to floor raft	No - structure demolished Yes - cement mortar
C6	Garage	In situ	Spalling to soffit	Yes
C7	Multi-storey Brewery	Concrete encased steel frame in situ floor	Spalling to soffit	Yes - gunite
C8	Flats	In situ	Spalling to soffit	Yes - gunite

APPENDIX C (continued)

Ref No.	Type of Building	Type of Construction	Damage	Repaired ?
C9	Warehouse	Waffle slab on RC columns	Spalling to soffit	Yes/no - demolished waffle, cols repaired
C10	Warehouse	Precast portal frame	Collapsed	No
C11	Flats	In situ hollow pot	Plaster spall	Yes - replace broken pots
C12	School	In situ frame	Spalling to soffit	Yes - gunite
C13	Coach station	Concrete encased steel frame	Spalling to column + soffit	Yes - gunite
C14	Warehouse	Portal	Spalling to frame	Yes - hand applied epoxy
C15	Car park	Waffle	Spalling to soffit	Unknown - gunite suggested
C16	Office block	In situ + woodwool slab	Spalling to cols burnt woodwool	Yes - gunite
C17	Barn	Portal	Spalling	Yes - hand applied mortar
C18	Car park	In situ	Spalling to soffit	Yes - gunite
C19	Flats	In situ + omnia floor	Severe spalling	Yes - gunite
C20	Brewery	In situ	Spalling to soffit	Yes - gunite
C21	Retaining wall	In situ	Spalling to bottom of wall	Yes - gunite
C22	Office	Spanform	General spalling	Recommend repair
C23	Flats	In situ	Spalling to soffit	Yes - gunite
C24	Supermarket	In situ frame	Spalling to soffit some expansion	Yes - hand applied mortar
C25	Car park	In situ RC	Limited spalling	Yes - hand applied mortar
C26	Squash court	Precast lightweight concrete walls	Slight crazing timber gallery destroyed	Yes
C27	Warehouse	In situ	Spalling to ground floor soffit	Yes - gunite

* Replies to Appendix B questionnaire not yet received from consultants concerned.

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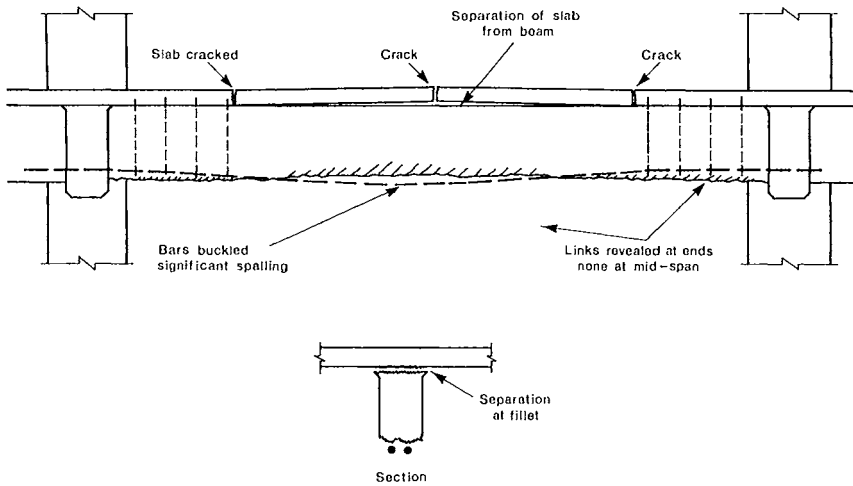


Fig. 1—Effect of unrestrained bars and slab

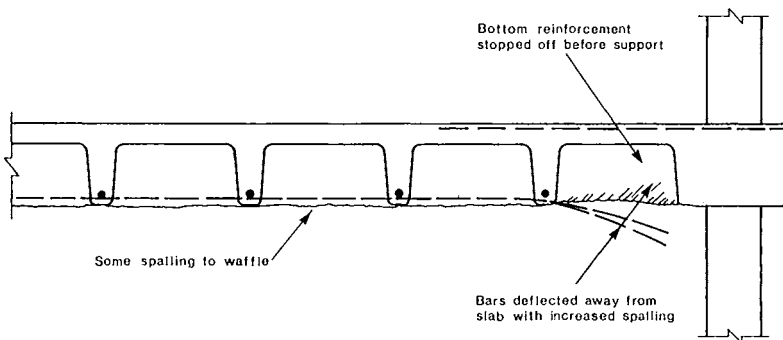


Fig. 2—Effect of non-support of bars at end of waffle slab

Fires During Nuclear Power Plant Construction

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Synopsis: In some cases, it is necessary to evaluate and document structures which were subjected to fires during construction. Due to the lack of official regulations on the subject, the analyses and documentation on these incidences, when reported to the regulatory authorities, implied unconventional activities, both for the utility and the A/E.

The paper describes several fires affecting nuclear power plant concrete structures. They were all caused by inadvertent human actions. The presence of highly combustible auxiliary construction materials contributed to their propagation.

It is recommended that the cost (purchase and installation) of these auxiliary materials be evaluated against the use of alternate (noncombustible) ones before making any decision, sufficiently in advance to avoid costly and time-consuming changes that may affect a usually tight schedule.

As a result of visual inspection, followed by "in situ" and laboratory tests and engineering analyses of the affected elements, some elements had to be demolished and reconstructed while others were found to be acceptable.

Keywords: acceptability; bond (concrete to reinforcement); compressive strength; concrete construction; evaluation; fires; fire tests; impact hammer tests; modulus of elasticity; nuclear power plants; tensile stress; tests; ultrasonic tests

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INTRODUCTION

Two types of fires which occurred during the construction of Nuclear Power Plant (BWR and PWR) during 1983 and 1984 are described in this paper.

The fires occurring in the BWR type Nuclear Power Plant (Fig. 1) corresponded to the ignition of the vertical formwork materials between walls (Fig. 2), due to incandescent particles, coming from welds or oxyacetylene cuts, falling on them. The fire was produced in the junctions between buildings when the second wall which forms the junction was being prepared for concreting. The fires caused a local elevation of the temperature in the concrete of the wall already built and in the reinforcement steel bars of the wall that was going to be concreted. The aim of the analysis was to ascertain if the materials of the affected areas were still suitable from the engineering and design viewpoint or if they should be demolished and replaced.

The fire which occurred in the PWR Nuclear Power Plant in 1983 affected the Electrical Building (Fig. 5), where the formworks between the 7th and 8th floors burned. As a result of the visual inspection, the analysis was aimed at determining the areas to be demolished.

ANALYSIS OF THE BWR FIRES BETWEEN WALLS

The analysis of the fires which occurred between building walls consisted in the following activities:

Description of the Incident

- F-1 At 16 h 25 m the fire was detected (Fig. 1) and use was made of the powder fire extinguishers. At about 16 h 45 m water hoses were used to extinguish the fire, which was accomplished at 18 hours. The cause was attributed to the oxyacetylene cutting of reinforcement steel.
- F-2 Water was directed from the roof of the Auxiliary Building (Fig. 1) at 21 h 49 m. Due to various impediments in the water supply the fire was not extinguished until 23 h 40 m. Water continued to be hosed until 2 o'clock the next morning. The cause of this fire was not clearly identified, although it was assumed due to welding operations.

Visual Inspection

Visual inspection of the fires indicated the same conclusions in both cases, although of greater dimensions in the second. Spalling of the concrete up to 15 mm deep was observed, cracks were not seen.

Only superficial concrete spalling resulted in the most affected areas. As the specified minimum protective covering of 40 mm had been reduced locally to still admissible values, the aim of the analysis was to substantiate that the constructed work subject to this incident still complies with the project requirements.

Drilled Sample Test

Table 1 summarizes the results of the drilled concrete and reinforcement steel samples tested from the affected and unaffected areas. Discrepancies in the strength properties of the materials were not observed. The safety margins between the tested and required materials strengths were sufficient after the two fires.

In Situ Tests

The results of in situ concrete tests with the sclerometer are included in table 2 (SCHMIDT hammer, type N). A large number of measurements were taken to suitably cover all the disturbed area. The results confirmed a local superficial concrete spalling, because the test results changed substantially with repeated hammer strikes.

In figure 3, the statistical correlation between sclerometer readings and ultimate concrete strength is given. This study used a sample of 290 concrete test cylinders, and each sclerometric measurement is the average of six readings. The test specimens were 3, 7, 28 and 45 days old.

The statistical interpretation indicates that for a minimum sclerometric value of 32 corresponds to an average ultimate concrete strength of 25 N/mm² with a 90% level of confidence. The specified concrete compressive strength was 25 N/mm².

The propagation speed measurements, done by PUNDIT device, are summarized in table 2. The test samples which broke with ultimate concrete strengths between 32.7 and 44 N/mm² had a direct propagation speed between 3.48 and 3.86 Km/s. Comparing these results with 3.42 to 3.78 Km/s in situ propagation speed (Table 1) is a procedure for qualifying the ultimate concrete strength in the fire exposed structure.

Simulated Fire

Five reinforced concrete test samples were prepared to carry out the bond test according to EH-80 (8). A brick wall was constructed with four of these test samples embedded, subjecting one of the faces to a simulated fire equivalent to the one under study. A thermocouple was placed to measure the temperatures reached at a depth of 15 mm from the face exposed to the fire. The test sample not subjected to the fire was used as a reference.

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Water spray was applied to the simulated fire, as in the live fires, but not in a continuous manner in order to not extinguish it. Not all the structure that was exposed to the live fire had the same water spray treatment. The bond test results are considered representative, because the heat and temperature reached in the contact between concrete and reinforcement were comparable in the specimen and in the real structure.

The results of the simulated fire were:

Result of thermocouple reading

Fire duration: 26 min

Temperature at the start: 10.9° C

Maximum temperature: 101.4° C after 19 min

Recovery

after 29 min: 93.5° C

after 2 h 49 min: 41.6° C

after 19 h 13 min: 10.4° C

The results of the bond tests are summarized in table 3 and are acceptable according to code EH-80 (8).

CONCLUSIONS

It is concluded that the fire temperatures and heat in the affected concrete areas did not reduce the overall and local structural capacity of the reinforced concrete walls.

It is worth mentioning that, in view of the test results obtained and of the pertinent visual inspection, superficial damage only occurred in the concrete in isolate areas and at depths limited to 15 mm. This fact is not considered preoccupying from the structural viewpoint as a minimum covering of 40 mm is provided for the rebars. Furthermore, the bond tests carried out to verify if there was resistance capacity loss in the concrete covering the rebars has made clear the valid behavior of the former.

The estimation of maximum temperatures reached in the concrete during fires is based on:

- a) The maximum reading obtained in the thermocouple located at 15 mm from the face exposed to the fire in the simulated test was 101.4° C.
- b) From the compared results of the modulus of elasticity that indicate a decrease to the order of 10%, and in view of graph A. 25 of ACI-216-12-81 (11), it is estimated that the average temperature reached in the concrete is around 100°C. This same value can also be reached following the indications of Peter Smith (9) or of P.V. Sullivan and Meller P. Poucher (10).
- c) From the concrete color change, which was detected in the extracted test samples and which affected a depth of not more than 10 mm, it is gathered that the temperature reached could be around 300° C, although it may even be assumed that said coloring could have been caused by the effects of the chemical components of the wood panels used as part of the formwork on burning, due to which it could even be considered that this temperature of 300° C had not been reached.



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- d) In the ultrasonic tests in fire F-2 the samples most affected by low speeds correspond to the hollow spaces with 343° and 333° azimuth, with a relative average speed of 0.86. Using this datum in figure 4 it can be assumed that the concrete temperature did not exceed 170° C.

It is concluded that the maximum concrete temperature due to fires did not exceed 300° C. There is some evidence that the temperature may not have exceeded 170° C.

Concrete test

No estimate of the concrete temperature during the fire can be reached from the concrete test sample data. However, the observed concrete compressive strength from these tests exceeds the specified design minimum.

Steel test

The tensile yield and ultimate strength for reinforcement bars exposed to fire were tested and their results were above the specified minima. Some steel test showed reduced strain values at rupture, or a loss of ductility.

ANALYSIS OF THE FIRE IN A PWR

The fire affected the Electrical Building during the construction of a PWR Nuclear Power Plant in 1983. This building was to be completely constructed (slabs, walls and columns) of reinforced concrete. The concrete used rolling siliceous aggregate, with a water cement ratio of less than 0.5 and 375 Kg/m³ of cement. It consists of 8 floors (two of which are basements) and a roof on which there are two penthouses. When the fire started the 7th floor was complete, as well as the walls and columns between the 7th and 8th floors. Likewise, the formwork and reinforcement for the 8th floor slab were in place.

The fire started during the early morning from a small fire used by the laborers to keep warm. The forms of the floors were prefabricated aluminum frames with phenolic linings. The wall and column forms were agglomerated panels supported by wooden beams glued with resin. The nearness of the small fire to the forms and their combustibility facilitated propagation of the fire. The fire lasted approximately two hours. The absence of workers because of a change of shifts delays in its detection. The fire, affected approximately a quarter of the surface of the 7th floor, that is 400 m². Practically all the forms installed in this area burned.

An inspection was made to determine the extent and the magnitude of the fire damage.

- The fire affected approximately a quarter of the 7th floor, and an area between the 7th and 8th floors. The forms for the 8th floor with the reinforcement placed had completely burned. This reinforcement was badly distorted, smoked, and showed slag and fly ash deposits. The interior wall and columns were greatly affected by the fire. The concrete protective covering detached and exposed the reinforcement.

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The exterior walls, on the inside faces, showed similar damage. Their external faces were slightly damaged.

The 7th floor slab fire damage was limited to two areas of spalling which exposed the reinforcement and another area with only superficial concrete damage.

Melted parts of the aluminum formwork were found, which indicated that concrete temperatures of over 600° C were reached.

Fly ash and vitreous element deposits were found on the reinforcement for the 8th floor. Their subsequent bond to the concrete could not be trusted. Furthermore, the fire caused deformations in the reinforcement, in accordance with the codes and standards, made them not suitable for reuse.

However, mechanical tests of these rebars were carried out to determine the elastic limit, obtaining acceptable results in all cases except for the rebars of diameter less than 16 mm.

To quantify the effects of the fire on the reinforcement, tensile strengths tests were carried out on the 16 mm diameter bars taken from the 7th floor slab. The strength and ductility properties were acceptable according to the test results.

It was decided to replace all the 8th floor slab reinforcement, and accept the reinforcement in the walls and columns (all greater than 16 mm). All other bars that remained in the concrete after the fire were also be considered acceptable.

It was observed that the wall and column concrete with considerable fire exposure between the 7th and 8th floors was in a precarious state, completely cracked, and decomposed. The repair would have meant chipping off the concrete surface to reach sound concrete, and repairing the walls with epoxy resin. This meant considerable cost and difficulty, with uncertainties about the structural behavior of complete repairs. The state of the construction permitted easy demolition and, therefore, this type of solution was not justified. Consequently, the problem was limited to defining the limits of the areas to be demolished.

The areas to be demolished was based on the determinations of the concrete compressive strength, the elastic modulus and on the bond between reinforcement and concrete.

The first two characteristics were obtained extracting concrete test samples. These were laboratory tested and the results compared with the specified values.

In situ reinforcement bond tests were carried out, trying to reproduce as close as possible the Spanish standard EH-80 for the bond test. Such testing was performed on probably acceptable elements (basically walls between the 7th and 8th floors).

For the bond tests, wall reinforcement bars were chosen (see Fig. 6), and cut or isolated at a certain distance from the upper surface of the wall (60 cm).

The concrete was chipped off the isolated bar, except in a length 10 times the bar diameter. A load was applied to the bar by

a hydraulic jack placed on top of a pair of beams resting against the upper surface of the wall and against an auxiliary metallic support. Load phases were applied measuring the bond stresses corresponding to displacements of 0.01 mm, 0.1 mm and 1.0 mm. The results were compared to the requirements of standard EH-80 (8) bond tests.

Furthermore, six tests were performed on reinforcement welds (tensile and bending) used for those bars without sufficient length for overlapping. These results were compared to UNE 36088 (5).

The affected areas were determined to be demolished. Basically, the greater part of the walls, all the columns between the 7th and 8th floors of the area affected by the fire and the two concrete spalling areas on the 7th floor slab.

EXPERIENCE SUMMARY

The steps followed for the qualification of the structures subjected to fires were:

Visual Inspection

- . Provides general idea of the damage.
- . Establish documentation test.
- . Prepare a precise description of the fire: Geometry and affected areas; Duration; Burned materials.

In Situ Test

- . Concrete and reinforcement samples.
- . Sclerometric measures for the superficial damage.
- . Propagation velocity measures, if possible through thickness.
- . Bond test.

Laboratory Test

- . Samples taken from affected areas shall be tested and compared with quality control test results.
- . Quantitative analysis of sclerometric measures with ultimate concrete strenght.
- . Propagation velocity measures as a qualitative interpretation of the ultimate concrete strength.
- . Simulated fire for bond test.

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4. Variation of relative propagation velocity with temperature
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TABLE 1
VERTICAL CONSTRUCTION JOINT FIRES SUMMARY ON SAMPLES TEST

			AFFECTED ZONES		NON AFFECTED ZONES
			F-1	F-2	
CONCRETE (1)	CONCRETE STRENGTH (Reference 1, 2, 3 & 8)	NUMBER	6	18	SPECIFIED LABORATORY TEST=25 TEST SAMPLE = 42.55
		MAXIMUM	51.22	60.84	
		MINIMUM	37.89	29.23	
		MEAN	45.44	43.80	
		STD. DEVIATION	4.56	9.38	
	MODULUS OF ELASTICITY (TANGENT) (Reference 1, 2 & 4)	NUMBER	1	7	MEAN LABORATORY TEST AT 360 DAYS 23501 TEST SAMPLE = 25212
		MEAN	22987	23600	
REINFORCING BARS (Reference 5, 6 & 7)	NUMBER OF TEST		7	54	SPECIFIED VALUES 420 550 18
	MEAN YIELD STRESS		516	490	
	MEAN ULTIMATE STRESS		687	688	
	MEAN ULTIMATE STRAIN (%)		18	19	

NOTES: ALL UNITS ARE N/mm^2 , EXCEPT ULTIMATE STRAIN IN %
 (1) 60 mm DIAMETER SAMPLES, LENGTH BETWEEN 86 AND 130 mm



TABLE 2**VERTICAL CONSTRUCTION JOINT FIRES SUMMARY IN SITU TEST**

		AFFECTED ZONES				NON AFFECTED ZONES
		F- 1		F- 2		
		P. (1)	NP. (2)	P.	NP.	NP.
SCLEROMETER (1)	NUMBER	134	171	196	196	119
	MAXIMUM	55	54	55	54	51
	MINIMUM	36	20	21	20	38
	MEAN	44.50	34.20	42.18	37.05	42.47
	STD. DEVIATION	4.84	6.05	5.67	6.51	2.90
PROPAGATION VELOCITY (Km/s)	NUMBER			9		—
	MAXIMUM	—		3.78		—
	MINIMUM	—		3.42		—
	MEAN	—		3.50		—
	STD. DEVIATION	—		0.17		—

P. PREPARED SURFACE

NP. NON PREPARED SURFACE

(1) HAMMER REBOUND

TABLE 3
EH- 80 BOND TEST

BOND STRESS

EH-80 REQUIRED VALUES	FOR $\varnothing=32$ mm	$t_{bm} = 4.16$ N/mm ²
	FOR $\varnothing=32$ mm	$t_{bu} = 6.92$ N/mm ²

TEST RESULTS

NON AFFECTED BY FIRE SAMPLE	LEFT INDICATOR	RIGHT INDICATOR
	$t_{bm} = 7.79$ N/mm ² $t_{bu} = 12.13$ N/mm ²	$t_{bm} = 9.03$ N/mm ² $t_{bu} = 13.76$ N/mm ²
SUMMARY OF RESULTS , MEAN VALUE OF FOUR SAMPLES SUBJECTED TO SIMULATED FIRE	LEFT INDICATOR	RIGHT INDICATOR
	$t_{bm} = 8.38$ N/mm ² $t_{bu} = 13.70$ N/mm ²	$t_{bm} = 8.36$ N/mm ² $t_{bu} = 14.03$ N/mm ²

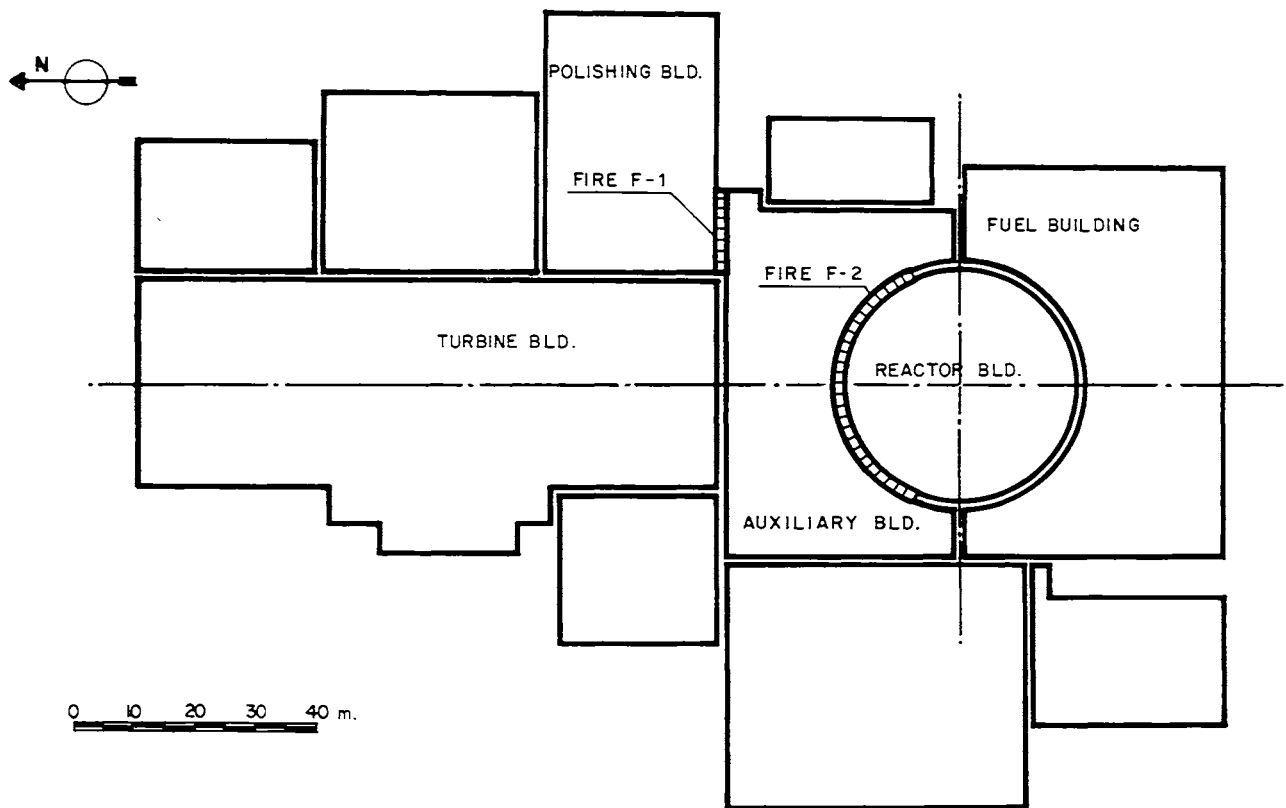


Fig. 1—BWR plot plan—affected zones by fires

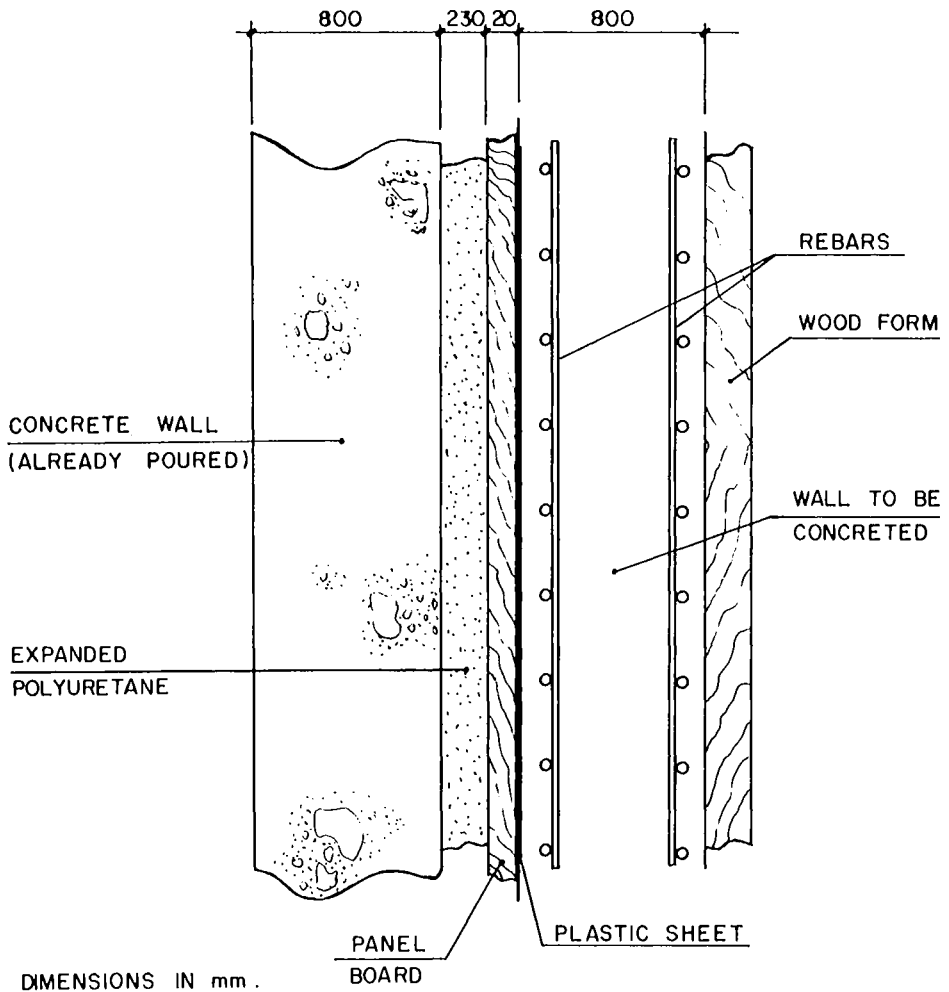
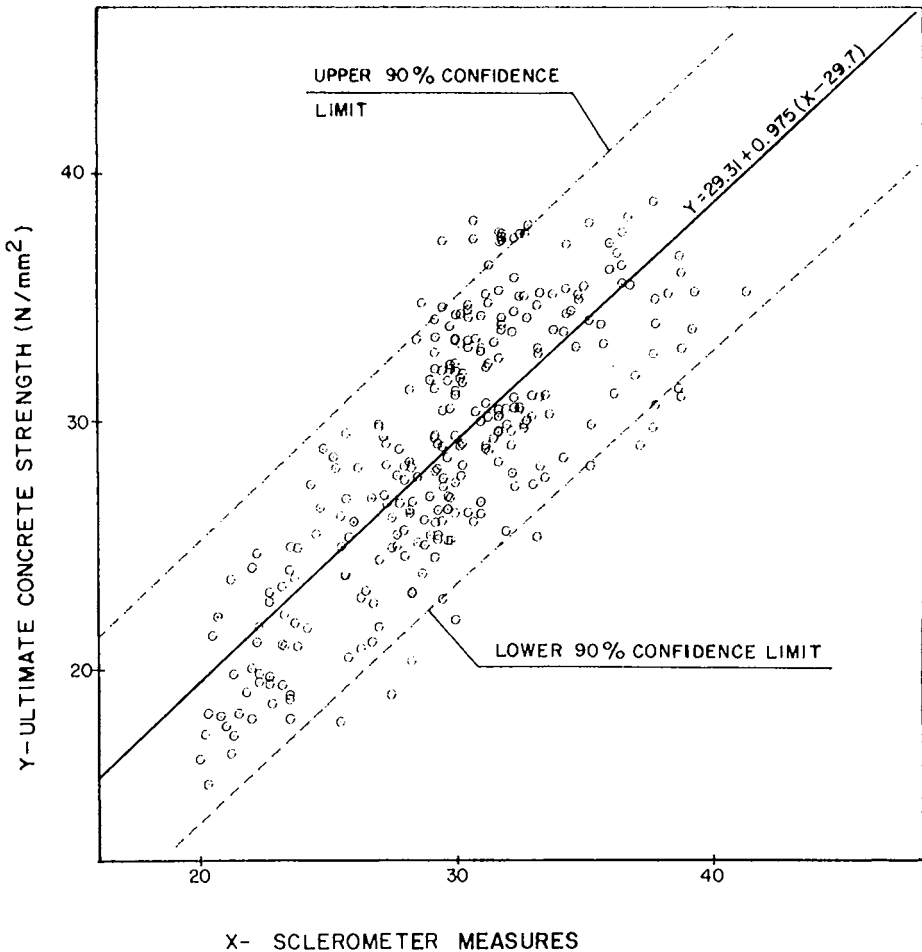


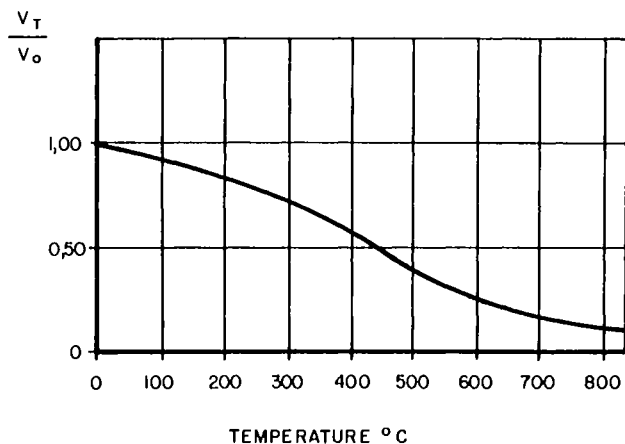
Fig. 2—Typical section between walls with forms



CORRELATION COEFFICIENT = 0.43

HIGHLY SIGNIFICANT FOR 1% LEVEL OF SIGNIFICANCE
(SNEDECOR STATISTICAL METHODS, 1940)

Fig. 3—Correlation between ultimate concrete strength and sclerometer measures in laboratory samples



— BY LOGOTHETIS (12)

v_o : INITIAL PULSE VELOCITY

v_T : PULSE VELOCITY AFTER HEATING

Fig. 4—Rate $\frac{v_T}{v_o}$ as a temperature function

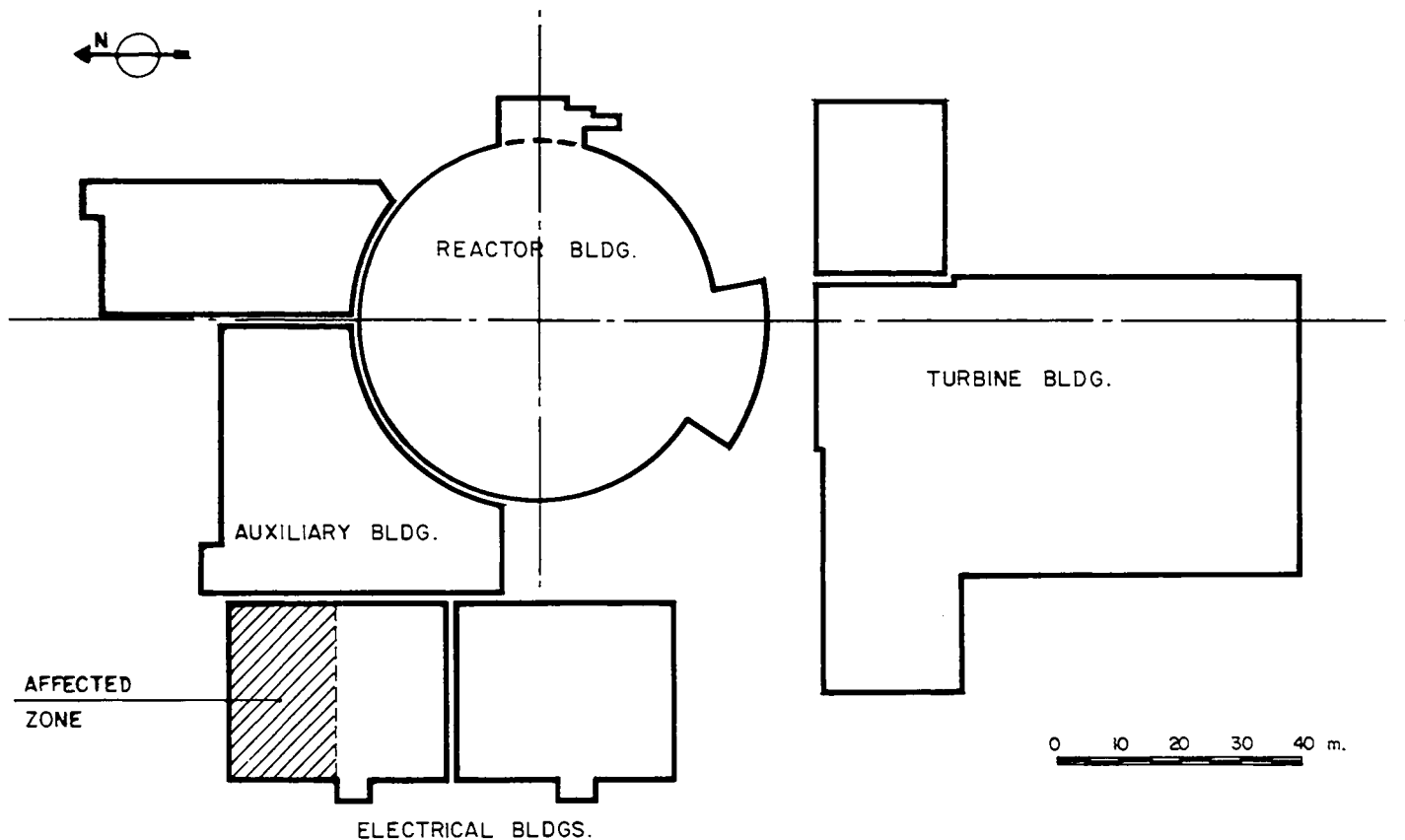


Fig. 5—PWR plot plan—affected zone by fire

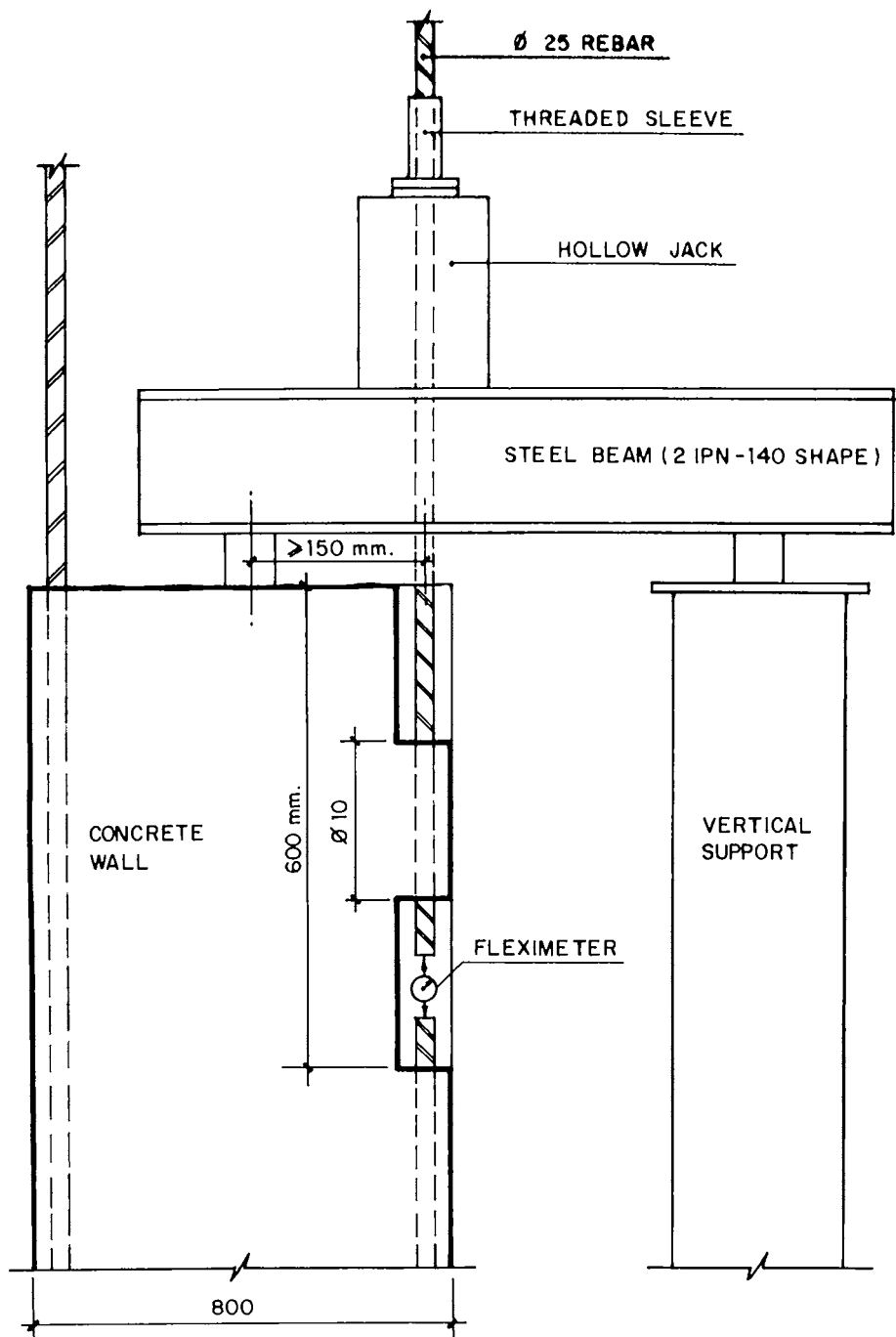


Fig. 6—In situ bond test

Old Flames Speak Well of Young Concrete

by Peter Smith

Synopsis: A major three span highway bridge in Southern Ontario was involved in a construction fire while it was being widened 25 years ago. Concrete, ranging in age from 6 days to 20 years, in an abutment, the adjacent arch rings and spandrel columns was severely damaged. The structure is now revisited to assess the long term performance of the remedial works and to consider whether the investigation and repairs might have taken a different course if today's testing, analytical and repair techniques had been available then. The strengthening of the damaged arch springings and the shotcrete restored concrete sections exposed to a harsh environment have stood the test of time. The review of investigative and repair methods identifies advances made in recent years, and justifies long term confidence in older procedures which are still in use.

Keywords: arch bridges; concrete construction; damage; fire resistance; fires; nondestructive tests; performance; repairs; shotcrete

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INTRODUCTION

On March 24, 1959 a construction fire, fueled by straw insulated wooden formwork and timber falsework, seriously damaged the reinforced concrete arches and abutment of a highway bridge that was being widened, Fig. 1 shows the scene on the morning after.

The existing bridge carried a major freeway over the Bronte Creek, some 40 km West of Toronto, Ontario, therefore the first imperative was to determine if this 20 year old structure could be reopened safely to traffic. Once this had been accomplished, attention turned to investigating the extent and structural significance of the damage with a view to remedial work. Naturally, the young concrete in the new arch and abutment which, depending on the location was only 6, 19 or 40 days old, was of special concern.

The first surprise in the investigation was the lack of published information on the effect of fire on young concrete. A knowledge base had to be assembled from information on mature concrete exposed to fires and fire tests and interpreted with considerable judgement. Next, investigative methods had to be selected and used to probe the condition and properties of the concrete and steel that remained, apparently intact, in order to analyze what could be salvaged by restoration of the section or which elements needed strengthening or replacing. Lastly, specific remedial works had to be devised, specified and executed which would ensure a durable structure with a long term performance in no way inferior as a result of the fire.

All these steps are described in detail in a paper presented at the time (1). The purpose of this paper is to revisit the bridge to determine if the diagnosis made, and the remedial work undertaken over 25 years ago have stood the test of time. In light of this experience, advancements in knowledge of the effects of elevated temperatures on materials, in investigative techniques and in repair methods are reviewed to see whether any of the advancements would warrant a different course of action if a fire of similar consequence occurred today.

LONG TERM PERFORMANCE OF THE REPAIRED CONCRETE

Nature of the Repairs and Subsequent Exposure

The investigation determined that the fire damage was more serious than evident from the obvious surface cracking and spalling of the concrete and the deformation of exposed reinforcing steel alone. Concrete, at least to the depth of the main reinforcing steel and especially on the vertical faces of the arches, was found to be unsound. Fortunately, the main steel and the core of concrete in both the old and new arch rings had been affected little by the fire. Figs. 2 and 3 are indicative of the extent of the damage and the concrete section requiring restoration in the 20 year old arch and the 19 day old arch ring respectively. These arches were the structural members of greatest concern as to their integrity and the serviceability of repairs. Ref. 1 contains detailed plots of the test results and condition surveys which determined the extent of repairs required and upon which the decision to proceed with repairs was made.

Except at the arch springings, where very deep spalling occurred, the 20 year old and the 6 and 40 day old concrete in the more massive abutment was only superficially damaged since formwork, which was still in place, did not fully burn. Formwork also afforded some protection to the underside of the new arch. As detailed in Fig. 15 of Ref. 1, the arch springings were repaired by encasement in a single concrete pedestal.

After removal of loose and suspect concrete, both the young and old concrete sections remaining were rebuilt with shotcrete. Following final cleanup by sandblasting, a small gauge wire mesh was attached to the existing concrete by ramsets. The shotcrete was then formed against edge strips and lightly trowelled to the intended section and 25 mm minimum cover over the reinforcing steel. The shotcrete was then wet cured for 7 days. As shown in Figs. 4 and 5, the work was carried out with care for quality and appearance.

The 19 day old concrete was about 3 months old when repaired and it has been continuously in an open, south-facing exposure, including many cycles of freezing and thawing, ever since. Parts of the substructure have been subjected also to brine runoff from deicing salt applied to the highway above each winter. As a result of adding the new arch, the repaired 20 year old concrete became one of the inner arches that were protected to some extent from the weather. The common strengthening of the old and new arch springings seen in Fig. 5 was later backfilled and protected from the elements.

Recent Maintenance Work

No appreciable maintenance was required to the bridge until routine inspections during the late 1970s showed general deterioration to the deck slabs, supporting beams, bearing seat areas and parts of the arch rings and spandrel columns. Damage was much more severe in the, by then, 40 year old original bridge than in the new widenings, and appeared similar to that in comparable reinforced concrete bridges exposed to the effects of freezing and thawing and deicing chemicals for many years. The upper surface of the deck was repaired, waterproofed and repaved in 1982. The work on the structure below the deck surface (where the fire had occurred) was undertaken in the following year. After removing deteriorated concrete and cleaning the reinforcing steel, latex-modified shotcrete or mortar was applied to restore the section and provide at least 25 mm cover to the steel. In addition, significant cracks were either epoxy grouted or, if subject to movement, sealed.

Surveys of Performance

The condition of the original repairs to the fire damaged concrete at the time of the 1977-78 inspections may be judged from Fig. 6. The outer arch ring and column stubs in the foreground, which contained the 19 day old concrete and were at the heart of the fire, were found not to require further repairs. An area of the old inner arch and a column (showing between the second and third columns from the right of the foreground arch), required repair. This concrete, located down the arch from Core No. 20 in Fig. 13 of Ref. 1, was the only significant area of repaired fire damaged concrete scheduled to be repaired again. This localized damage seemed entirely due to brine leakage channeled from above. Fig. 7 shows the 1983 repair work in progress on this part of the old arch. Some areas of concrete which had been exposed to fire, but not damaged to the point of repair, did require repair in 1983. An example is the right hand column of the old inner arch shown in Fig. 6. But since similar deterioration was prevalent in other old spans that had been well clear of the fire, delayed effects of the fire are unlikely to have contributed to the corrosion spalling.

Late in 1984 a further visual inspection, with soundings, was made of the bridge substructure affected by the fire. No apparent or latent defects were found. The arch in the foreground of Fig. 8 contained the concrete that was only 19 days old at the time of the fire. The general integrity of the concrete arch and the condition of the 25 year old shotcrete repairs appear to be excellent. A similar observation applies to the repaired concrete in the exposed abutment which was 6 days old at the time of the fire. The column in the background of Fig. 8 is one of those in which the concrete was 20 years old at the time of the fire but which had been repaired recently because of corrosion spalling. Unlike the original shotcrete

repairs, the new latex-modified shotcrete, here and elsewhere in the structure, displays significant surface shrinkage cracking.

The strengthening pedestal to the arch springings could not be examined because it is buried. No structural cracking or settlement has been observed in the arches or abutment, so it seems reasonable to deduce that this remedial measure has also stood the test of time.

REVIEW OF ASSESSMENT AND REPAIR TECHNIQUES USED

The remaining question is: If the construction fire had happened today, would anything have been done differently in light of experience and new knowledge?

The first admission is that the references cited in the original paper (1), though still useful, need updating. References 2 through 16 provide more recent sources on material properties, testing, structural behaviour and fire damage assessment. Ref. 4, 8 and 16 in particular contain bibliographies, or lead to other references, that may be helpful in a particular situation.

Comparison of Testing and Investigating Methods

The investigation of the construction fire damage to the Bronte Creek bridge in 1959 used a then state-of-the-art panoply of inspection and testing techniques. These were:

1. Visual inspection to identify and locate spalls, cracks, exposed reinforcing steel, fused materials and other evidence of the duration and temperature of the fire and the severity of damage.
2. Petrographic examination of the concrete, from both loose and core samples, to identify colour change, dehydration, microcracking, loss of bond to aggregate or steel, autogenous healing, etc.
3. Physical testing of concrete cores for residual compressive strength and of cut samples of reinforcing steel for tensile strength.
4. Non-destructive testing using ultra-sonic pulse velocity measurements and Schmidt Rebound Hammer readings.

Fortunately, both fire damaged and unaffected concrete of the same class and age were available in different parts of the structure so that direct comparisons were possible. This greatly increased the confidence in the data, especially that from measurements of pulse velocity.

In the intervening years many of the procedures outlined above have been refined in light of experience and new knowledge (2), (3), (4), (5), (6). Improved testing equipment has made site work a lot easier; for example, ultra-sonic testers have been miniaturized and now have easy-to-read digital displays. Confidence in interpreting data has increased; for example, that from the two non-destructive tests used (7). In some instances alternative devices are available; for example, the Windsor Probe in place of the Schmidt Hammer. But proven new methods that might have influenced or radically changed the course and outcome of the 1959 investigation are few.

Some newer, non-destructive tests (8), (9), have potential for assessing the effects of fire on reinforced concrete. These tests include; impulse radar and infrared thermography (to detect discontinuities at right angles to the detector), acoustic emissions (to detect deformation and stress under changing load) and radiographic or radiometry methods (to give an internal picture or measure density). However, all of these require elaborate equipment, site access may be a problem, and interpretation of the data is not an established routine. Other new tests, such as determination of indentation microhardness or plucking of steel tendons have been used in assessing damage to prestressed concrete structures only (3), (4), (6).

Twenty-five years ago, the duration and intensity effects of a fire had to be judged by such minimum temperature indicators as colour change and quartz spalls in the concrete or by identifying fused materials. Since data on the residual properties of concrete and steel after heating to various elevated temperatures (and quenching) were available; the amount of core drilling, pulse-velocity work and laboratory testing might have been reduced, if more reliable means had been available to establish the temperature regime to which the concrete, in various locations, had been subjected. Subsequently, Harmathy (10) proposed a method based on thermogravimetric and dilatometric testing of samples, secured within a day or two of the fire, to determine a temperature history inwardly into the concrete. Another approach is the residual thermoluminescence of quartz method reported later by Smith and Placido (11), which they state gives an estimate of thermal exposure (time/temperature) that can be correlated with residual concrete compressive strength. Only a small sample is required that can be extracted easily from loose concrete or a core that is suspect.

Material and Structural Considerations

The striking resistance of immature concrete to fire damage displayed at the Bronte Creek bridge has been amply confirmed by other incidents; one of which involved concrete in a building that was less than 2 days old (12). Yet the paucity of knowledge as to why this should be so, in face of low

strength and high moisture content in the concrete, still remains. But then, alternative opinions still remain to be resolved on the mechanism of spalling, which is such an obvious feature of fire damaged concrete of any age (13). As already referenced, significant information, has been published over the last quarter century about the behaviour of mature concrete and structural elements and assemblies exposed to high temperature, including the beneficial effects of continuity and restraint from cooler parts of the structures. With care, much of this information can be extended to young concrete.

The structural elements affected by the fire at Bronte Creek were essentially compression members and hence the possibility of loss of bond strength between concrete and steel was not considered as seriously as might have been in tension members. Furthermore in 1959, unless dehydration at the interface was evident, it was not considered that significant reduction in bond was likely (1). More recent work (14), (15) tends to show this judgement was naive, and that possible reduction in bond efficiency should always be questioned after a fire (15). The adequacy of bond with epoxy coated reinforcing bars, used on constructions that will be exposed to deicing chemicals, has raised concern in some minds. Fire could be an added detriment. At the time of writing (June 1985), full scale fire tests, which should resolve this doubt, are about to start at the Portland Cement Association laboratories.

The task of analyzing the load carrying capacity of a fire damaged structure, has become easier in recent years. In addition to a better understanding of the effects of elevated temperatures on reinforced and prestressed concrete, improved structural analysis techniques and the calculating power of the computer are now on command. Notwithstanding, proof load tests still may be called for in some critical situations to settle ultimate doubts.

Review of Repair Techniques

The need to repair damage to bridges due to reinforcing steel corrosion and to rehabilitate other concrete structures, has led to many new or modified materials or techniques in recent years. For example, the recent repairs to the Bronte Creek bridge used latex-modified shotcrete and mortar. The attributes of latex-modified shotcrete are reviewed in Ref. 17. Other modifications to shotcrete to improve strength and adhesion and to reduce shrinkage and rebound, such as the addition of steel fibres and silica fume, are possible but they have not yet been widely used. Notwithstanding these developments, the performance of the less expensive, conventional shotcrete originally used to repair the fire damage, could hardly be bettered.

The importance of removing all unsound concrete and achieving a clean substrate to aid good bond cannot be over-emphasized. For these purposes, high pressure water jet cutting in replacement of jack hammering and sandblasting is a promising development.

The secrets of successful concrete restoration work have not changed over the years. It is desirable to match the properties of the new work as closely as possible with those of the old. It is essential to provide a sufficient depth of low permeability concrete cover to any reinforcing steel in order to inhibit corrosion for as long as possible. Meticulous preparatory work and application of the new concrete or mortar are mandatory. The results of these prescriptions and wet curing to develop strength and reduce initial drying shrinkage are evident in the long term durability of the shotcrete repairs at Bronte.

Speaking generally, no new breakthrough in materials or methods, proprietary or generic, has yet appeared to displace shotcrete (guniting) as the first choice for restoring fire damaged concrete sections (2), (3). Nor is there any substitute for good workmanship in sight.

CONCLUDING REMARKS

Clearly there are gaps in knowledge about the fire resistant properties of concrete at early ages still to be filled. Improved investigative and repair techniques will always be welcome. Yet, those faced with the consequences of a construction fire may take heart from this feedback report in the validity of past and present practice.

Once again, concrete has demonstrated that it is the Phoenix of construction materials, even when immature. Irrespective of the age of the concrete, knowledge and techniques sufficient to determine the extent of the fire damage, assess structural adequacy and devise remedial measures have been proof tested by time and all too frequent use. More often than not, standard concrete repair methods can restore the new structure to full serviceability at less cost than replacement. Judged by performance in a highway bridge exposed to a harsh environment, the life expectancy of such remedial work is at least 25 years. All provided, of course, that the best of engineering knowledge and judgement, concrete practice and workmanship govern.

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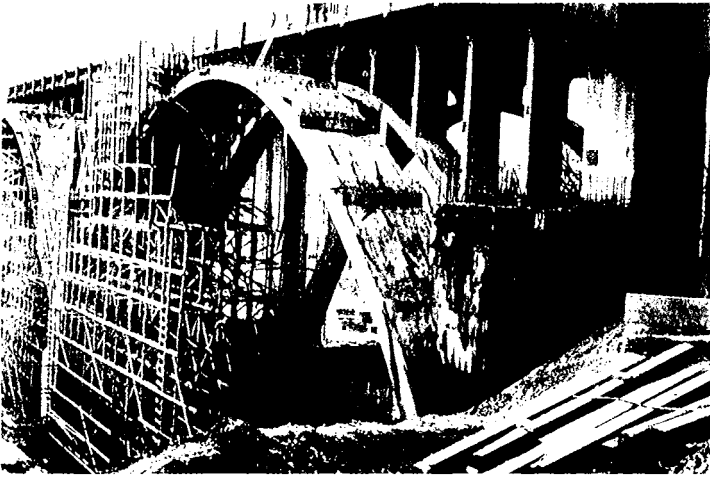


Fig. 1—Scene after the fire (1959)



Fig. 2—20-year-old concrete arch prior to shotcrete repair (1959)



Fig. 3—Preparatory work on the 19-day-old concrete arch (1959)

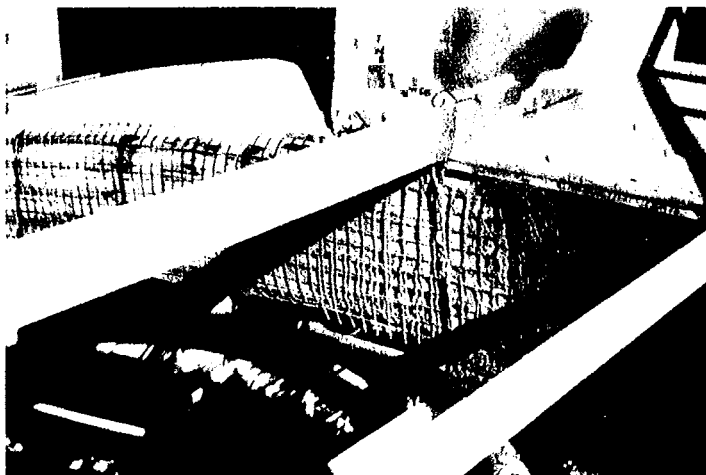


Fig. 4—Shotcrete application and finishing in progress (1959)



Fig. 5—Completed structure showing part of the repairs and strengthening (1959)



Fig. 6—Condition of the repaired structure after 18 years (1977)

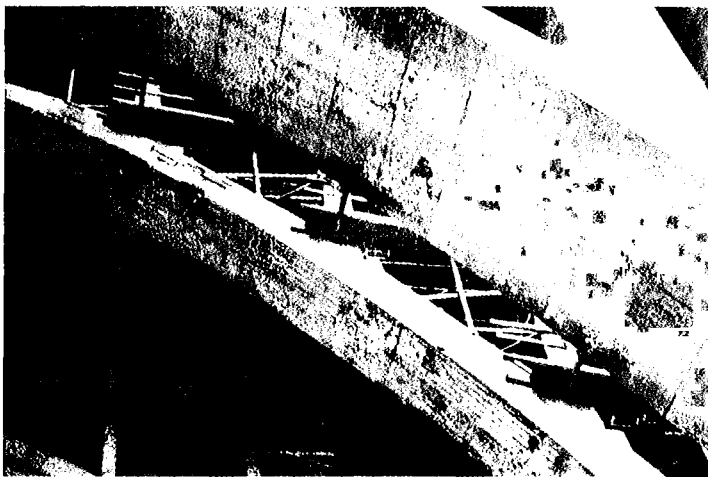


Fig. 7—Repair work in progress on the old inner arch (1983)

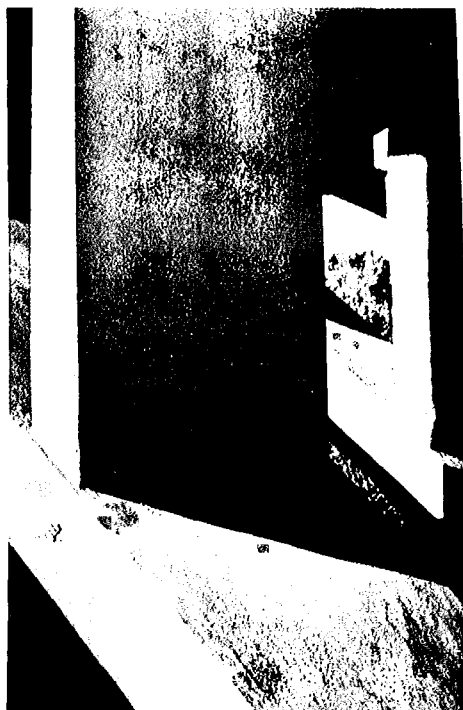


Fig. 8—Condition of shotcrete repairs after 25 years exposure (1984)

Assessment and Repair of Fire-Damaged Concrete Structures—an Update

by A. K. Tovey

Synopsis: The purpose of this paper is to outline some of the proposed changes to the Concrete Society Technical Report 15 which is the most comprehensive UK document dealing with assessment and repair of fire-damaged concrete structures. The publication is being updated to take into account changes of practice and methods of assessment since 1978. Repair techniques other than Guniting are to be included with particular comment in respect to resin repairs.

Keywords: buildings; concrete construction; damage; estimates; evaluation; fires; plastics, polymers, and resins; repairs; shotcrete

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INTRODUCTION

A number of publications (1-4) within the UK advise on the assessment of the effect of a fire on concrete structures. The most comprehensive, with regard to concrete structures, is the Concrete Society Technical Report 15 (1), "Assessment of fire-damaged concrete structures and repair by Guniting". That publication is currently being revised by the Fire Resistance Committee of the Concrete Society UK and this paper outlines the major revisions. It is expected that the revised report will be available mid-1986.

The principal objectives have been to update the information given in the light of changes of practice and methods of assessment since 1978 and to include repair techniques other than Guniting (sprayed concrete). This has led to the change of title to "Assessment and repair of fire-damaged concrete structures".

The revised Report is split into three main sections - 1. Assessment, 2. Design, 3. Repair methods. The detailed section on assessment follows that given previously; a general qualitative assessment followed by a quantitative investigation of individual members. A flow chart (Figure 1) has been added to assist in developing a programme for assessment.

QUALITATIVE ASSESSMENT

The general qualitative assessment method gives a simple way to indicate the degree of damage and likely repair of individual members and overall structure. Figure 2 shows part of a first survey on a four-storey warehouse. The number adjacent to each member indicates the visual degree of damage according to the classification system given in Technical Report 15 (1) which is to be standardized to make it simpler to use (Table 1). A detailed first assessment of this type enables a very good initial picture to be drawn of the overall condition of the structure and the likely repair.

An example is given in Figure 3 which shows the whole of the first and second floors of the part structure used in Figure 2.

This clearly shows that the main damage was sustained in the upper left-hand quarter of the first floor with the most notably affected members in the area bounded by columns 5, 8, 19 and 22. The degree of damage is again clearly shown to reduce away from this area towards the lower right-hand corner (col.59). This corner was the position that the fire started but the major combustible material was located to the rear of the building adjoining a single-storey steel framed warehouse which was totally destroyed. The fire can be seen to have spread to the second floor but was brought under control before a major fire or extensive damage had developed. A comprehensive report giving a good indication of both damage and likely repair can be prepared in only a few days using the simple visual assessment procedure. At this stage, any surprising aspect in the performance of a structure would also be noted.

QUANTITATIVE ASSESSMENT

The detailed quantitative investigation using various tests is to determine the likely temperatures to which the member has been subjected and hence the loss, if any, in strength of reinforcement and concrete. From examination of the building contents, it is possible to assess the maximum likely fire compartment temperatures and, from eye witness reports and evidence of combustible content, to assess the equivalent standard fire period. This can be compared with the temperature distribution and strength loss determined from test evidence. Report 15 (1) indicated three main tests which could be carried out in order to determine strength loss in concrete. These are: color determination, Schmidt hammer and core testing. The revised report deals with these and, in addition, gives guidance on the use and selection of other tests such as ultrasonic pulse velocity (5), Windsor probe (5), BRE internal fracture (5) and thermoluminescence tests (6). The information obtained from tests is then used to assess the strength of the member. An example of tests carried out on a structure is shown in Figure 4. This figure shows the position and results of a series of different tests on a section of wall and floor slab. Each of the results can be compared together to improve the confidence of the assessment. The pundit readings on the floor, for example, show the degree of damage from the maximum occurring at the seat of the fire (readings around 200) to that where the concrete is undamaged (readings around 34) and hence indicate the area requiring repair.

The strength reduction curves given in the previous report were based on research on unstressed concrete (7). It is known that the strength reduction of concrete unstressed at the time of heating is greater than for concrete under compression. Since the majority of structural concrete will be stressed (at least under dead load) at the time of heating, then some modification to the previous data is appropriate. The strength loss data has been

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simplified to a straight line plot for consistency with other documents (8,9) (Figure 5).

The color of concrete can change as a result of heating (10) and therefore may be used to indicate the maximum temperature attained and the equivalent fire duration. In some cases at above 300°C a pink discoloration may be readily observed. The onset of noticeable pink discoloration is important since it coincides approximately with the onset of significant loss of strength due to heating. Therefore, any pink discolored concrete should be regarded as being suspect.

Pink discoloration is due to the presence of ferrous salts in the aggregate and/or in the sand, and in some cases these are not present. Therefore, concrete which has not turned pink is not necessarily undamaged by fire. The color change to pink tends to be more prominent with siliceous aggregates. Calcareous and igneous crushed rock aggregates are less susceptible to this effect.

The residual strength of reinforcement can be judged from the assessed temperature of the surrounding concrete or, if spalling has occurred, from the period of exposure and likely fire compartment temperature.

The effect upon steel during elevated temperatures and after subsequent cooling has been researched in detail (11,12). Significant loss of strength may occur while the steel is at high temperature and this is usually responsible for any excessive residual deflections. However, recovery of yield strength after cooling is generally complete for temperatures up to 450°C for cold worked steel and 600°C for hot rolled steel. Above these temperatures, there will be a loss in yield strength after cooling. The actual loss in strength depends upon the heating conditions and type of steel but the simplified values given in Figure 6 will be sufficient for most purposes. Where this aspect is critical to the assessment, the matter should be discussed with the reinforcement manufacturer if known or, alternatively, tests carried out on samples taken from the member. Loss in ductility may occur after exposure to particularly high temperatures. Buckling of reinforcing bars often occurs as a result of compressive stress induced at high temperatures by restraint against thermal expansion.

A damage factor may be determined for each member although normally it will only be necessary to consider Class 3 and Class 4 damaged members in detail. The damage factor is determined from Figure 5 using the estimated average temperature within the compression block obtained from temperature profile curves as, for example, are given in Figure 7.

The general process for this is to record the depth of the layer which has reached a temperature of 300°C by reference, where appropriate, to the depth of the pink zone. Estimate the increase