IMPORTANCE OF STRUCTURAL FIRE PROTECTION IN REINFORCED CONCRETE BUILDINGS: A CASE STUDY OF EX-NASACO BUILDING IN DAR ES SALAAM

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Reinforced concrete is concrete that is internally reinforced with steel bars or mesh. This gives the material the compressive strength of concrete along with the tensile strength of steel. Reinforced concrete does not perform well under fire conditions as it spalls and loses strength. Heating causes a failure of the bond between concrete and steels bars and this results in reduction of strength of concrete. Strength of concrete decreases with increase in temperature and increase with time. In order to provide fire resistance to structural components/elements in buildings, it is important to consider the type of aggregates to be used, to adhere to the recommended sizes of structural components/elements and to provide adequate concrete cover. Also, provision of concrete or masonry walls and partitions reduces the spread of fire within the building.

Keywords: *Reinforced concrete, structural fire protection, fire resistance, concrete cover, concrete strength, compressive strength.*

INTRODUCTION

For many years fire has been considered to be one of man's greatest aids to his advancement as it gives him a source of both light and heat. However, if fire is not strictly controlled it can be one of the greatest hazards man has to face. Fire can and does break out in all forms of buildings and one of the main causes is faulty electrical equipment and wiring. Therefore, provision of fire protection for the building elements is required under statutory provisions or in connection with insurance requirements. The regulatory control for fire protection is concerned with safeguarding the occupants in the building where the fire may occur, minimizing risk to the adjacent buildings and ensuring the stability of the building (Chudley, 1999).

Over years, structural fire protection has developed into a concept with five main objectives namely:

- Preventing the initiation of fire,
- Restricting the growth and spread of fire and prevent a building from becoming unstable,
- Containment of fire within specified boundaries – a compartment forming part of the building or the whole building,
- Means of escape for the occupants of the building , and
- The control of fire by automatic devices and by active fire fighting.

The fire protection objectives are realized by specifying requirements for passive as well as active measures. Passive measures are part of the built system and are functional at all times and include building layout, design and construction.

Of the five main objectives mentioned above, the second has occasionally been considered to be the sole aim of structural fire protection, probably due to the method of expressing requirements. Retention of stability is essential to achieve the other objectives, as it allows an easier control of fire and reduces the possibility of a large and destructive fire.

Fire protection requirements include specifications for fire resistance of buildings. Although the control is concerned with the behaviour of the buildings as a whole, the requirements apply to individual structural elements of constructions. The assumption is made that if the individual structural elements are satisfactory when together (as in the building) the whole building should perform at least as well.

This paper is about the appraisal of structural stability of the fire gutted EX-NASACO building in Dar es Salaam.

EX- NASACO BUILDING

The construction of EX – NASACO fire gutted building was completed in 1985 and 10 years later on 17th August 1995 the building was gutted by fire. The building is a frame structure of reinforced concrete with a basement, mezzanine and seven floors. The framed structure consists of columns, beams, floor and roof slabs, staircases and a central lift shaft.

The concrete cover for structural components are as follows: 20 mm for slabs; 25 mm for beams and 30 mm for columns. Average dimensions for structural members are 400 mm x 600 mm for columns, 250 mm x 500 mm and 300 mm x 600 mm for beams and 150 mm for floor slabs.

The overall stability of the building is provided by the lift shaft and stair wall structure. The building being of in situ reinforced concrete frame construction had excellent shear and bending moment distribution qualities. In situ reinforced concrete roof and floor slabs as well as stair cases at the ends of both wings provide additional stability. The building had timber partitions and plastic sun-shading devices. The design strength for beams and slabs was 20 N/mm^2 and for columns was 30 N/mm^2.

EFFECTS OF FIRE ON CONCRETE STRUCTURES

Introduction

Concrete is a porous substance bound together by water-containing crystals. When heated to high temperatures the binding material can decompose. The loss of moisture causes shrinkage to the concrete, and the rising of the temperature causes the aggregates to expand leading to cracking and spalling of the concrete. Spalling of concrete from the exposed surfaces can seriously affect the load - bearing capacity of structural members especially if it occurs during the early stages of exposure. The effect of temperature on compressive strength of concrete is shown in Figure 1 (Whinnet, et al. 1978).

For temperatures up to 300 ^{0}C , the strength of structural quality concrete is not significantly reduced. However, after repeated cycles up to this temperature, progressive strength loss may be expected. At temperatures greater than 500 ^{0}C, a significant reduction of compressive strength may occur. Damage arising from the effects of high temperatures normally occurs in the form of spalling.

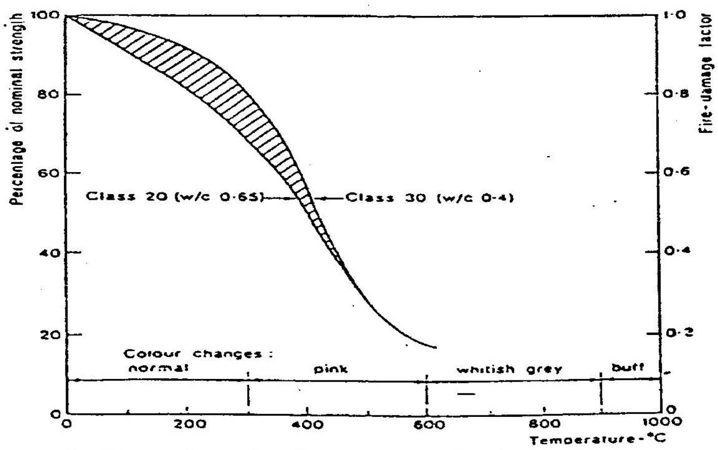

Figure 1: Strength and color changes for dense concrete

A summary of the possible effects that can occur both during heating and cooling are given in Table 1.

Fire Resistance

Fire resistance of a structural member is its protection for a certain period of time which it is anticipated will give sufficient delay to the spread of fire, ultimate collapse or noticeable change in the rate of deformation of a structure, time for persons in danger to escape and to enable firefighting to be commenced.

Table 1: Effects of fire attack on reinforced concrete (Chudley, 1999)

Stage	Probable effects
On heating	
1. Rise in temperature	Surface crazing
2. Heat transfer to interior concrete	Loss of concrete strength, cracking and spalling
3. Heat transfer to reinforcement	Reduction of yield strength; possible buckling and/or deflection increase
On cooling	
4. Reinforcement cools	Recovery of yield strength appropriate to maximum Temperature attained; any buckled bars remain buckled.
5. Concrete cools	Cracks close up; reduction in strength until normal temperature is reached; deflection recovery incomplete for severe fire; further deformations and cracking may result as concrete absorbs moisture from the atmosphere.

Reinforced concrete structural members have good fire resistance properties and being non-combustible do not contribute to the spread of flame over their surfaces. It is possible however, under the intense and prolonged heat of fire that the bond between the steel reinforcement and the concrete will be broken resulting in spalling of concrete and hence reduction of strength. Therefore, there is a need to provide adequate fire resistance for reinforced concrete structures, which depends mainly on the thickness of structural members, the provided concrete cover, duration of fire attack on the structure and the type of aggregate used. Experience shows that for many constructions requiring a fire resistance of not more than two hours the sections normally used to satisfy the loading requirements may possess sufficient fire resistance.

Thickness of structural members:

BS 8110 provides minimum thickness necessary for various structural members to meet the specified fire resistance in hours. Fire resistance of structural members increases as the size of the members increases (see Table 2).

Concrete cover

Steel reinforcement, because of its behaviour under fire, requires a protective cover varying in nature and thickness according to the standard of fire resistance [Foster, 1988]. Concrete cover has to provide protection to the reinforcement from fire and environmental attack. The concrete cover described in this context refers to the distance between the nearest heated face of the concrete and the surface of the main reinforcement. The recommended concrete

Table 2: Minimum dimensions of reinforced concrete members for fire resistance [BS8110, 1985]

Fire resistance	Minimum beam width (b)	Rib width(b)	Minimum thickness of floors (b)	Column width (b)			Minimum wall thickness		
				Fully exposed	50% exposed	One face exposed	ρ<0.4%	0.4%<ρ< 1%	ρ>1%
H	mm	mm	Mm	Mm	Mm	Mm	mm	mm	Mm
0.5	200	125	75	150	125	100	150	100	75
1	200	125	95	200	160	120	150	120	75
1.5	200	125	110	250	200	140	175	140	100
2	200	125	125	300	200	160	-	160	100
3	240	150	150	400	300	200	-	200	150
4	280	175	170	450	350	240	-	240	180

ρ is the area of steel relative to that of concrete.

covers and the corresponding fire resistance are shown in Table 3. The fire resistance of reinforced concrete structural members increases as the concrete cover increases.

Type of aggregate

Type of aggregate has significant effect upon the thermal properties of concrete. Limestone aggregate concrete has a coefficient of thermal expansion of about half that of quartzitic-aggregate concrete; lightweight aggregate provides a material less likely to spall if it has low internal moisture content, and which has a better temperature strength characteristic and lower thermal conductivity.

Concrete and Steel

Structural concrete

The strength of concrete varies with temperature changes. The strength of dense aggregate concrete decreases when the temperature reaches 300^0 C, while for lightweight concrete its strength decreases when temperature reaches 400^0 C (Mpinzire, 1997).

Structural steel

Structural steel used is either hot rolled or cold worked conditions. The use of structural steel in concrete enables lighter structures to be built by using relatively smaller sized members which provide adequate strength, stability and durability. Steel reinforcement is used in concrete for the following purposes;

- To take the tensile stresses in concrete beams or slabs
- To withstand shear stresses in beams which are greatest near the supports. These give rise to complementary tensile stresses in such regions which require the use of additional reinforcement – either in the form of stirrups or bent-up bars.
- To carry a proportion of the compressive stresses and to withstand tensile stresses which may arise due to eccentric loading, as in columns.
- Reinforcement may be used near the surface of mass concrete structure to control cracking due to drying or carbonation shrinkage.
- In some cases, secondary reinforcement is used to prevent spalling of concrete surface due to fire, etc. The structural steel will continue to serve the above purposes if it is safe from chemical, physical and fire attacks. But when steel is heated to a certain temperature level, it undergoes changes in its chemical and physical properties. In BS 8110, it is stated that reinforcement steel changes its properties after reaching 300 ^{o} C while pre-stressing wires or strands change just at 150 ^{o} C temperature. For example, the strength of structural steel becomes 50% of that at 20 ^{o} C after reaching just above 550 ^{o} C for reinforcing steel and 400 ^{o} C for pre-stressing wires or strands with further

Table3: Nominal cover to all reinforcement (including links) to meet specified periods of fire resistance [BS 8110, 1985)

Fire resistance	Nominal cover						
	Beams		Floors		Ribs		Columns
	Simply supported	Continuous	Simply supported	Continuous	Simply supported	Continuous	
H	mm	mm	mm	mm	mm	mm	mm
0.5	20	20	20	20	20	20	20
1	20	20	20	20	20	20	20
1.5	20	20	25	20	<u>35</u>	20	20
2	<u>40</u>	30	<u>35</u>	25	**45**	<u>35</u>	25
3	**60**	<u>40</u>	**45**	<u>35</u>	**55**	**45**	25
4	**70**	**50**	**55**	**45**	**65**	**55**	25
Cases that are bold require attention to the additional measures necessary to reduce the risk of spalling (BS 8110: Part 2)							

reductions as the temperature increases above these figures. Depending upon the proportion of the design loading applied during the fire, this loss at high temperatures is usually responsible for any excessive residual deflection. However, recovery of yield stress after cooling is generally complete for temperatures up to about 700° C for mild steel and hot- rolled high- yield. 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 temperature by restraint against thermal expansion.

ASSESSMENT OF DAMAGE

General

When a fire has occurred, the requirements are generally for an immediate and thorough appraisal carried out with clear objectives. Such an appraisal must begin as soon as the building can be entered and generally before the removal of the debris. The fire resistance of a concrete structure is frequently well above its minimum requirements and because of the structural continuity present in most buildings, it is normally higher than is required for the structure to survive fires and be reinstated.

Investigations

Initial Investigation

Initial investigation was carried out to establish the extent of damage to the building. It was established that the fire did not affect the basement and ground floors, affected the mezzanine floor minimally, but devastated the first to seventh floors. The fire gutted structural members were classified depending on the extent of damage (Hutchison, 1995). Seriously fire damaged members had damage class 3. Extensive fire damage was due to combustible materials for partitions, openings and timber furniture. It was revealed that none of these members exhibited severe fire damage class 4. This was so mainly because of the following:

i) The fire was relatively short-lived and was well ventilated. The very high temperatures and consequent damage associated with fires involving chemicals, and the like in confined buildings were therefore not reached,

ii) Plaster rendering, much of which spalled off during the fire, provided some additional protection to the reinforced concrete members,

iii) The concrete aggregate is crushed coral which has a coefficient of thermal expansion approximately half that of quartzites. Fire resistance was therefore, better than with other aggregates.

Structurally, the integrity of the building remained intact because of the reasons stated above and, the central core comprising lift shaft and stair wall survived the fire more or less unscathed. The building appears to be well founded with no evidence of foundation settlement.

Principal Investigation

The principal investigation was carried out for structural members with fire damage class 3 to check if the deformation is within allowable limits for the design of the structure and to establish the residual strength of concrete. The principal investigation covered load test and cutting cores for strength tests. The loading test showed that the deflections were lower than the allowable elastic deflection (Mpinzire, 1997).

Cutting cores

The main reasons for drilling the cores samples were as follows:

i) To determine the colour changes of the concrete and fire severity,

ii) To determine the residual strength of concrete slabs and beams with damage class 3 (Hutchison, et al., 1995).

Visual assessment of the cores

The quality of concrete was assessed from the size and quality of visible voids and cracks on the surface of concrete. Only one core out of the eight drilled had a horizontal crack thus indicating poor workmanship during construction and a reduction of load-bearing capacity of the structural member. There was no significant colour change observed and from Figure 1 it can be concluded that the reduction in strength is not so significant.

Strength tests

After visual assessment, the cores were cut to size, capped and tested for compressive strength.

A correction factor according to the length/diameter ratio of the cylindrical specimen after capping was obtained from BS1881 to yield the estimated in situ cube strength. This strength was again modified to take into account the presence of reinforcement steel, to yield the estimated cube strength. The test results are shown in Table 4.

Table 4: Cubes compressive strength.

Core Indication	Concrete cover (mm)	Cube strength (N/mm^2)	Strength Reduction (%)
1st floor slab - 1	20	12.9	36
1st floor slab – 2	20	17.5	13
1st floor slab - 3	20	12.9	36
2nd floor beam - 1	25	15.2	24
2nd floor beam - 2	25	7.9	61
2nd floor beam - 3	25	15.2	24
3rd floor slab - 1	20	15.6	22
3rd floor slab - 2	20	17.0	15

Observations

From the results shown in Table 4 the following is observed:

i) The average determined cube compressive strength for floor slabs is about 15 N/mm^2. Some cores showed strength values slightly less than this due to local defects and unevenly distributed effects of fire.

ii) The average estimated cube compressive strength of the tested beams is about 15 N/mm^2. One core exhibited a lower strength due to the horizontal crack within the drilled and tested core.

iii) The tested third floor slab has an average estimated cube compressive strength of about 16 N/mm^2.

Heat penetration depth was on average between 20 – 30 mm. The maximum penetration depth observed was 45 mm.

Rebound Hammer Test

Test Results

In order to avoid damage to the columns, rebound hammer tests were carried out to predict the strengths of columns. For comparison purpose rebound hammer tests were also carried out for slabs and beams where cores were drilled and tested. The results of the rebound hammer tests are shown in Table 5.

Observations

The rebound hammer test determines in reality the hardness of the concrete surface, and

Table 5: Rebound hammer test results

S/N	Identification	Average rebound number	Standard Deviation
1	1^{st} Floor slab -1	23.6	1.8
2	2^{nd} Floor beam - 1	28.1	3.8
	2^{nd} Floor beam - 2	28.0	2.1
	2^{nd} Floor beam - 3	26.8	2.5
3	3^{rd} Floor slab - 1	31.4	5.8
4	Basement Column - 1	36.8	3.7
5	3^{rd} Floor Column - 1	37.3	2.8
	3^{rd} Floor Column - 2	37.1	3.3
6	4^{th} Floor Column - 1	32.5	3.4
	4^{th} Floor Column - 2	35.6	1.5

although there is unique relation between hardness and strength of concrete, empirical relationship can be determined for similar concrete. Changes affecting only the surface of the concrete, such as the degree of saturation and carbonation can give misleading results as far as the properties of concrete within the structure are concerned.

In this case, the rebound numbers obtained are compared to the results of core tests.

i) The concrete slabs showed a mean rebound number of about 24 in the first floor and 31 in the third floor, which is a strength of between 15 - 18 N/mm^2.

ii) The concrete beam in the second floor showed a mean rebound number of about 28 which is the strength of about 18 N/mm^2 .

iii) Columns showed a mean rebound number of about 34 in the third and fourth floors, and about 37 in the basement. This indicates strengths of between 20 – 25 N/mm^2.

CONCLUSIONS

From the above investigations and observations, the following can be concluded.

i) According to the minimum dimensions and concrete cover provided for different structural members, the design fire resistance was up to 3 hours.

ii) The maximum value for strength reduction observed for structural members was 36% hence indicating that the temperature was up to 300° C.

iii) Timber partitions and furniture, and plaster on reinforced concrete beams and columns were significantly affected.

iv) Fire damage was significant from the mezzanine floor and upper floors while the structural elements in the basement and ground floor were not seriously affected.

v) Fire spread was significant externally, while internally it was more confined due to the presence of concrete and masonry partitions, and floor slabs.

vi) From the damages observed, the main problem was expansion of the steel reinforcement causing spalling and falling down of cement plaster and concrete cover at some points. Still the building can be rehabilitated and strengthened.

RECOMMENDATIONS

Structural fire protection is generally a passive measure of fire protection, which is incorporated within the design specifications of each element of construction. The purpose of fire protection is to ensure that during a fire, the temperature of structural members or elements does not increase to a figure at which their strength would be adversely affected. For the design of fire protection, it is not possible to give an element or member complete fire protection in terms of time, therefore elements/members have to be given a fire resistance for a certain period of time which it is anticipated will give sufficient delay to the spread of fire, ultimate collapse of the structure, time for occupants in danger to escape and to enable fire fighting to be commenced. Therefore, structural members have to be designed in accordance to the specified fire resistance and the recommended concrete covers. A fire resistance of not less than 3 hours is recommended for reinforced concrete structures. Exposed combustible materials such as plastic sun shading devices are not recommended for construction of buildings.

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