8. *MITIGATION ASPECTS OF FIRE LOADS*

8.1 INTRODUCTION

Current building code requirements for determining the fire resistance of structural systems are based on the reaction of specimens to a standard fire exposure defined by international test standards of ASTM E119, ISO 834 and NFPA 251. Full scale fire tests with realistic loading are difficult and very costly as the building size grows. Thus mathematical simulation of the structural and thermal interactions become important, to supplement the tests. These numerical tools are formulated using finite elements, making them problem specific. In contrast most design codes allow simple procedures with empirical formulas for analysis under fire loads. Thus the need for performance based analysis becomes greater, so that the building is gauged on the basis of a certain level of performance it is expected to meet. To develop performance-based solutions that assure structural safety during fire conditions, the theoretical knowledge and technology of fire protection engineers must be integrated into the structural design process.

In this chapter, two compartments have been studied for their response to fire loads, in order to understand the effect of temperature load on 3-Dimensional R.C.C. frames. The first one is a corner compartment and the other an inner one. 5 load cases are applied where in the temperature load is applied from bottom slab to top with step wise application to columns and beams.

8.2 COMPARTMENT FIRES

The study of fire is done in context with the particular space it occurs in, commonly known as the compartment. Fire loads are calculated in terms of compartment loading.

A fire in a compartment will typically have three distinct phases as follows[107]:

1. ***Growth Phase:*** The fire is starting to grow from its point of origin and the temperature within the compartment is beginning to rise;
2. ***Fully Developed Phase:*** Flashover has likely occurred and the compartment and all of its contents are engulfed in flame; and
3. ***Decay Phase:*** The period during which the compartment temperature starts to decrease as the fire consumes all available fuel and begins to lose energy.

These phases are represented graphically in **Fig. 8.1.**

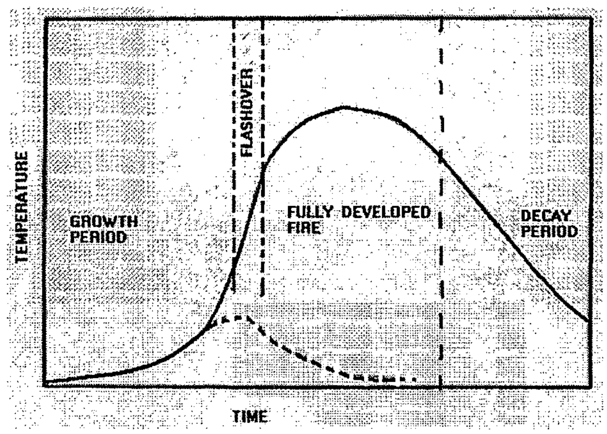

Fig 8.1. Typical Compartment Fire Temperature-Time Curve

A common definition of flashover is the point at which the radiant energy incident upon the floor of the compartment is 20 kW/m^2, and the temperature at the ceiling is 600 °C [94]. A compartment fire acheiving flashover is of importance as it is during the fully developed phase that room temperatures may be as high as 1100°C . The length of time these temperatures can be maintained will have a direct impact upon the structural integrity of the compartment, since the potential for structural damage is greatest when the temperatures are highest.

8.2.1 Ventilation Vs. Fuel Controlled Fires

The type of mathematical relationship that can be used to develop a time-temperature curve for the actual design fire is dependant upon whether the fire can be defined as ventilation or fuel controlled. A fire can be described as ventilation controlled when the burning rate is controlled by the available supply of oxygen necessary for combustion. Whereas a fire is fuel controlled when the burning rate is controlled by the availability of fuel, under a fully ventilated condition. The Fire Protection Engineering Handbook mentions that compartments with fuel loads ranging between 40 kg/m^2 to 100 kg/m^2 usually experience ventilation controlled fires. Furthermore, it states that a ventilation controlled fire is usually the most severe fire when analyzing a fire in a single compartment. This is the case because in a fuel-controlled fire the excess air entering the compartment is likely to have a cooling effect on the room temperature.

8.2.2 Room Fuel Load

One of the factors affecting the duration and intensity of the fire will be a function of the room fuel load. Therefore the first step in establishing the compartment fire time-temperature curve is to determine the room fuel load. The fuel load in a room is primarily made up of both fixed and moveable loads. The New Zealand building code suggests the following Upper Limits:

Residential Occupancy:	400 MJ/m^2 floor area
Office Occupancy:	800 MJ/ m^2 floor area
Retail Occupancy:	1,200 MJ/ m^2 floor area

Based on the summary results the variable fuel load in a typical office ranges from 330 to 800 MJ/m^2 per unit floor area. If these fuel loads are converted to a wood equivalent using an average heat of combustion of 18 MJ/kg for most woods, the fuel load can be converted to a range from between 18 kg/m^2 – 45kg/m^2. Although little data exists on the range of total fuel loads to be expected (variable plus fixed), the CIB W14 Study [94] suggests that the total fuel load in a typical office could range anywhere from 635 MJ/m^2 to 3900MJ/m^2 per unit floor area, which converts to 35kg/m^2 to 217 kg/m^2 per unit floor area.

8.2.3 Fire Load calculations

As such any method for calculating Fire Load has not been identified as a protocol in any of the codes on structural engineering. The following passage examines three methods from three different sources which appear to be on the same lines and give results close to each other.

8.2.3.1 Sweden survey

Traditionally, the combustible contents of compartments are qualified in terms of specific fire load which is defined as the mass of conventional combustibles (cellulosics) per unit floor area. If non-cellulosics are also present, their mass is converted into calorifically equivalent mass of wood by the application of multiplier ΔHf / ΔHw, where ΔHf is the heat of combustion of the non-cellulosic fuel and ΔHw that of wood. **Table 8.1** [96] shows some data on the statistical median of the specific fire load, Lm. They were calculated from the results of fire load surveys in Sweden.

Table 8.1 Fire Load in Calorifically Equivalent Mass of Wood

Occupancy	Lm (kg m^{-2})	Standard Deviation	L80 (kgm^{-2})
Dwelling	30.1	4.4	32.3
Office	24.8	8.6	30.0
School	17.5	5.1	20.9
Hospital	25.1	7.8	31.8
Hotel	14.6	4.2	17.7

It has been suggested in Sweden that the design of fire safety be based on the 80th percentiles, L80, in the applicable cumulative plots of specific fire load. When dealing with a building's key elements, it seems advisable to allocate some additional margin of safety. The design value of the specific fire load is expressed as a product of the statistical median, Lm, and a safety factor, s. An equivalent design fire load, Ge, can be calculated from the following formulae:

Ge = AF * L80 (For dividing elements) ....(8.1)

Ge = AF * s * Lm (For key elements) ....(8.2)

Where, AF is the floor area.

From the definition of the specific fire load that the actual design fire load, g, is equal to Ge only if the combustibles consist entirely of cellulosics. This being so, Ge can be regarded as representing the actual (assumedly all-cellulosics) fire load. If, however, the combustibles consist entirely or predominantly of non-cellulosics, the fire may assume entirely different characteristics. In calculating these characteristics, the actual design fire load, G, is the input information needed which is different from the equivalent design fire load, Ge.

The following equations apply:

G = Ge (If the combustibles consist wholly or predominantly of cellulosics)

$G = Ge * \Delta Hf / \Delta Hw$ (if the combustibles are non-cellulosics) ....**(8.3)**

8.2.3.2 Second approach : Fire as a building design load

The specification of appropriate design fires is a significant problem in the context of performance-based designs under either performance codes or under the equivalency clause in prescriptive codes. The interrelationships between structural design and fire protection make it highly desirable to find a way to treat fire as a building design load. A fire confined to the object of origin with radiant flux to other combustibles in the room of about 1 kW/m^2 is insufficient to drive flame spread. Similarly, a heat flux of 3 kW/m^2 would typically drive flame spread only near the object of origin. A heat flux of 15 kW/m^2 may ignite other objects in the room but is below flashover. A heat flux of 25 kW/m^2 is characteristic of flashover that would result in flames out the door and spread to the adjacent compartment. [97].

Table 8.2 Maximum Upper Layer Temp With Extent of Flame Spread

Extent of Flame Spread Class	Radiant Flux From The Upper Layer (kW / m^2)	Maximum Upper Layer Temperature (°C)
Confined to Object	1	100
Confined to Area	3	200
Confined to Room	15	450
Beyond Room	25	600

The fire load that the building is required to resist is defined as a thermal stress (steady-state upper layer temperature) resulting from a fire within a building space. The energy needed to produce the target temperature is a function of the geometry, heat losses, and ventilation of the space and can be estimated as shown in Eqn 8.4

$$. Q = \{\sqrt{g} * Cp * \rho o * To^2 * (\Delta T / 480)\ 2\}^{1/2} * \{hk * Aw * Ao * \sqrt{Ho}\}^{1/2} \quad ....(8.4)$$

$$= C * \{hk * Aw * Ao * \sqrt{Ho}\}^{1/2}$$

Where, Ao = Area of opening = Wo * Ho (m^2)

Aw = Compartment surface area minus area of opening

= 2(L * H + L * H + H * W) – Ao (m^2)

Cp = Specific heat of air (Cp = 1.0 kJ kg-1 K-1)

g = Gravitational constant (9.8 m/s^2)

hk = Effective heat transfer coefficient = k/δ (kW/m K)

k = Thermal conductivity of walls/ceilings (kW/m C)

δ = Thickness of walls/ceilings (m)

ρo = Ambient gas density (1.2kg/m^3)

T = Temperature (° C or ° K)

C = Value of Constant representing1st term; shown in **Table 8.3**

Table 8.3 Values of constant C

Upper Layer Temperature (°C)	C
100	38
200	130
450	480
600	750

Values in **Table 8.4** show that, the small event in an office-sized room about 50 kW is the typical trash can fire. The medium event at under 200 kW might be enough to ignite a nearby object by radiation but is unlikely to spread. At nearly 700 kW a large event will spread and the very large event at more than 1 MW would flash over the space.

Table 8.4 Fire Loads for Events in a Small Office

Event	Upper Temperature (°C)	Constant C	Maximum Q (kW)
Small	100	38	53
Medium	200	130	183
Large	450	480	676
Very Large	600	750	1057

8.2.3.3 Third approach for fire loads in office buildings (IIT-K)

The results of a fire load survey of office buildings in the city of Kanpur, India are used to give compartment fire loads. The survey covered a total floor area of about 11,720 m^2, covering 388 rooms in eight government office buildings with height up to four stories.The load survey was conducted for a year in 1992-93.

The term fire load is defined [98] as the heat energy that could be released per square meter of floor area of a compartment or storey, by the complete combustion of the contents of the building and any combustible parts of the building itself. Conveniently, the fire load is given by,

$$qc = (\Sigma\ mv * Hv) / Af \qquad ....(8.5)$$

Where, qc = Fire Load (MJ/m^2); Af = Floor area (m^2); mv = Total mass of the vth combustible material (kg); Hv = calorific value for the vth combustible material (MJ/kg).

The calorific values of different materials can be obtained from **Table 8.5** [98].as given in IS:1641 (1960)

The fire load falls into two general categories, viz., "non-movable contents" and "movable contents". The non- movable contents consisted of the combustible wall materials and covering; ceiling and floor finish materials; doors, windows and ventilators. Movable contents include combustible furniture; equipment and goods; and the combustible contents within and on top of items like metal furniture, containers etc. The mean and the 95^{th} percentile fire loads from the Indian survey are 348 MJ/ m^2 and 1030 MJ/ m^2 respectively. It was observed that fire load is not dependent on floor level of the building. In office buildings, wood and paper contribute to a substantial portion (98.7%) of total fire load. The movable combustible contents contribute to 88.3% of the total fire load.

Table 8.5 Calorific Values of common Combustible Materials

Materials	Calorific Value (MJ/kg)
Clothes (average)	18.8
Kerosene	37.2
Leather	18.6
LPG	49.9
Paper (average)	16.3
Plastic (average)	22.1
Rubber	39.5
Wood (average)	18.6

8.2.4 Prescriptive Codes

The limitations of most of the prescriptive codes are that they are directly based upon the standard ASTM E 119 fire exposure and its acceptance criteria for undamaged new construction. These standards have been the basis for determining Fire Resistance Ratings in building code applications since the 1920's. Although these standards have resulted in a reasonable level of safety, there is a growing body of evidence, which suggests that the entire testing procedure used by these standards is not realistic, in the modern construction era and the new building materials being used. Specifically, the time-temperature curves used by the standards do not compare to the time-temperature curve of a real compartment fire. The result is that building construction may be needlessly costly. These simple calculation methods and design aids were developed for single members or isolated subassemblies in a fire compartment subjected to a standard E 119 fire with idealized boundary conditions i.e., without overall structural system interactions and are not intended to be a predictor of structural failure in buildings for actual loads. The identification of specific ultimate structural failure modes or their potential progression to collapse mechanisms cannot be obtained from these formulations. However, these methods enable an efficient and generally conservative way to expand the fire resistance ratings for members and assemblies that do not directly match published tests.

8.3 PERFORMANCE BASED FIRE ANALYSIS

International Codes have introduced performance based provisions for fire safety as an alternative to existing prescriptive codes. However, due to the interrelationships in fire safety design, the role of the design fire that is larger than anticipated, the design of the structural system and its integration with the other fire protection systems, performance based design becomes an interdisciplinary and complex problem. Typically, these methods invoke additional structural mechanics and thermodynamic principles, more detailed analytical models, and more extensive computations. Exposed, unprotected members or assemblies, unique fire exposure conditions, new materials or designs, and existing or damaged construction may all exceed the typical scope of the prescriptive code criteria. A higher level of complexity will be encountered when a framing subassemblage, or the entire building frame, is to be directly analyzed for its structural response with applied loading effects under fire. Using factors such as fuel load and ventilation, the maximum credible fire in different locations in the building is calculated, and the structural response to these fires is evaluated. Calculating a structure's response to fire is a three step process [101]:

1. Fire Hazards Analysis to identify all credible fire scenarios and determine the impact of each scenario on adjacent structural members.
2. Thermal Analysis to calculate temperature history in each member.
3. Structural Analysis to determine forces and stresses in each member and whether local or progressive structural collapse would occur during any of the fire hazard scenarios.

8.3.1 Thermal Analysis

Structural members exposed to hot gases from fires gradually heat up and can reach very high temperatures. All thermal analyses start with discretizing the structural members into finite elements and defining boundary conditions, both fire-exposure boundaries and other boundaries where heat may escape from the member into adjoining parts of the structure or into the environment. The thermal material properties are defined for all components of the model, and the time-dependent fire curve from the particular fire scenario to be considered is specified. The equations are then solved to obtain the temperature history in all parts of the structural member during the fire. Such temperatures form the basis for a structural analysis of each member and the structure as a whole.

8.3.2 Structural Analysis

Once the maximum temperature loading in each structural member is known, calculations to determine the structural response of these members to the fire can be made, particularly to determine whether any member will fail during the fire. Standard structural analysis methods and computer models can be used, but they must take into account the special characteristics of materials at high temperatures:

- ◈ Thermal expansion (coefficient of expansion multiplied by temperature change), which can be very large in a fire. When there is restraint acting, very large stresses can be generated by this thermal expansion.
- ◈ Effect of temperature on material properties, such as modulus of elasticity, yield point, and ultimate strength. When steel becomes hot enough, the yield point can drop so much that the member cannot support gravity loads during the fire and collapse will occur. The degradation of yield strength with temperature for mild steel is shown in **Fig 8.2** [101]. It can be seen that between 1000°F and 1100°F (500°C-600°C) the yield point has fallen to only 60 percent of its room-temperature value.

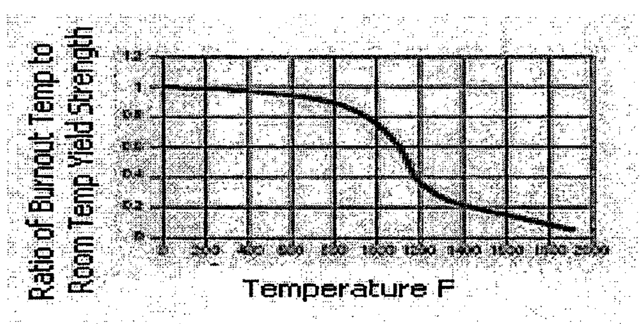

Fig. 8.2 Degradation of Steel Yield Strength

Concrete loses strength more slowly at elevated temperatures than steel does, but it is susceptible to spalling, which may expose reinforcing steel to fire and loss of strength.

- ◈ Nonlinear behavior. Structural response during a severe fire can quickly lead to high stresses, yielding, creep, and local or general failure. A complete analysis must take these nonlinear effects into account.

Several computer software are specifically designed to model these special high-temperature phenomena such as FIRES-RC II, FASBUS II and SAFIR. Other nonlinear structural programs such as ABAQUS and DIANA have been modified and utilized for fire analysis. General-purpose linear programs can sometimes also be used, particularly if member temperatures are not very high or if there is little restraint to thermal expansion. When using a linear program, the analyst must account for any yielding or other non-linear behavior by modifying material properties as the analysis proceeds.

8.4 EFFECT OF FIRE ON RCC ELEMENTS

Fire performance of concrete is generally good, considering its non-combustible nature and ability to function as a thermal barrier, preventing heat and fire spread. Since the thermal diffusivity is rather low, compared to steel, strong temperature gradients are usually generated within fire-exposed concrete members. Together with the high thermal inertia, this means that the core region may take a long time to heat up. Thus, the compressive strength of concrete is rapidly lost beyond a critical temperature. Structural effectiveness is not affected until the bulk of the material reaches the same temperature.

Concrete far from being a homogenous material, each of its components has a different reaction to thermal exposures in itself, and the behavior of the composite system in fire is not easy to define. It is common for design codes to bypass the complexities of temperature distributions by simply specifying a certain depth of concrete cover to the reinforcement bars in a composite structure, providing an insulating effect upon steel. So, the steel reinforcement in R.C.C. elements is not damaged until the concrete cover spalls off completely and fire reaches the bars.

Thus the two main criteria defined for resistance against fire of RC elements in all codes are:

- Large cover depth to all RCC elements.
- Wire mesh reinforcement can be provided in cover region to prevent spalling of concrete and thus fire cannot reach the main bars before the rating time.

Concrete can withstand temperatures up to 300° C without loss of strength. Significant loss of strength occurs only when the fire is sustained (4-6 hours) and the temperatures are in the range of 600 to 1200° C. As far as reinforcement is concerned, mild steel bars do not lose strength significantly at temperatures under 700° C. In high strength deformed bars there is a sharp loss of strength at 600 ° C (22% loss in ultimate strength, 42% in yield strength). At 1000° C the loss is 36% and 47% respectively [102].

One of the most complex characteristics in the reaction of concrete to high temperatures or fire is the phenomenon of 'explosive spalling'. Spalling is often assumed to occur only at high temperatures, yet it has also been observed in the early stages of a fire and at temperatures as low as 200°C. If severe, spalling can deteriorate the strength of reinforced concrete structures, due to enhanced heating of the steel reinforcement. Spalling may significantly reduce or even eliminate the layer of concrete cover on the reinforcement bars, leading to a reduction of strength of the steel and hence a deterioration of the mechanical properties of the structure as a whole.

Another significant impact of spalling upon the physical strength of structures occurs via reduction of the cross-section of concrete available to support the imposed loading, increasing the stress on the remaining areas of concrete. The mechanism leading to spalling is generally thought to involve high thermal stresses resulting from rapid heating and/or large build-ups of pressure within the porous concrete, which the structure of the concrete is not able to dissipate, due to moisture evaporation. These actions lead to the development of fractures and expulsion of chunks of material from the surface layers.

Thermal expansion and dehydration of the concrete due to heating may lead to the formation of cracks which is another phenomenon may provide pathways for direct heating of the reinforcement bars, possibly bringing about more thermal stresses and further cracking. In reinforced concrete structures, concrete and steel exhibit similar thermal expansion at temperatures up to 400°C; however, higher temperatures will result in significant expansion of the steel compared to the concrete and, if temperatures of the order of 700°C are attained, the load-bearing capacity of the reinforcement will be reduced to 20% of its design value.

8.5 COMPUTER IMPLEMENTATION USING SAP2000

The study of effect of temperature load on framed structures using SAP2000 software is done on two compartments, one on the outer corner and the other in the interior of an existing building have been considered.

8.5.1 Defining Temperature Load in SAP 2000

Two Load combinations have been considered

1) 1.5 (DL + LL) 2) 1.5 (DL + LL + Temper)

Temperature load case as shown in **Fig 8.3** has to be primarily defined. The rest of the steps are as shown below:

- Select the number of frame elements (beams and columns) to which the temperature loads have to be assigned.
- Go to 'Assign' menu and in that select 'frame/cable/tendon loads' as shown in the **Fig 8.4** and assign the 'Reference temperature' (room temperature) and the 'Temperature' command to assign the desired temperature value.
- In a similar manner the temperature loads for the slab members can also be assigned, by selecting the 'area load' command as shown in **Fig. 8.5**

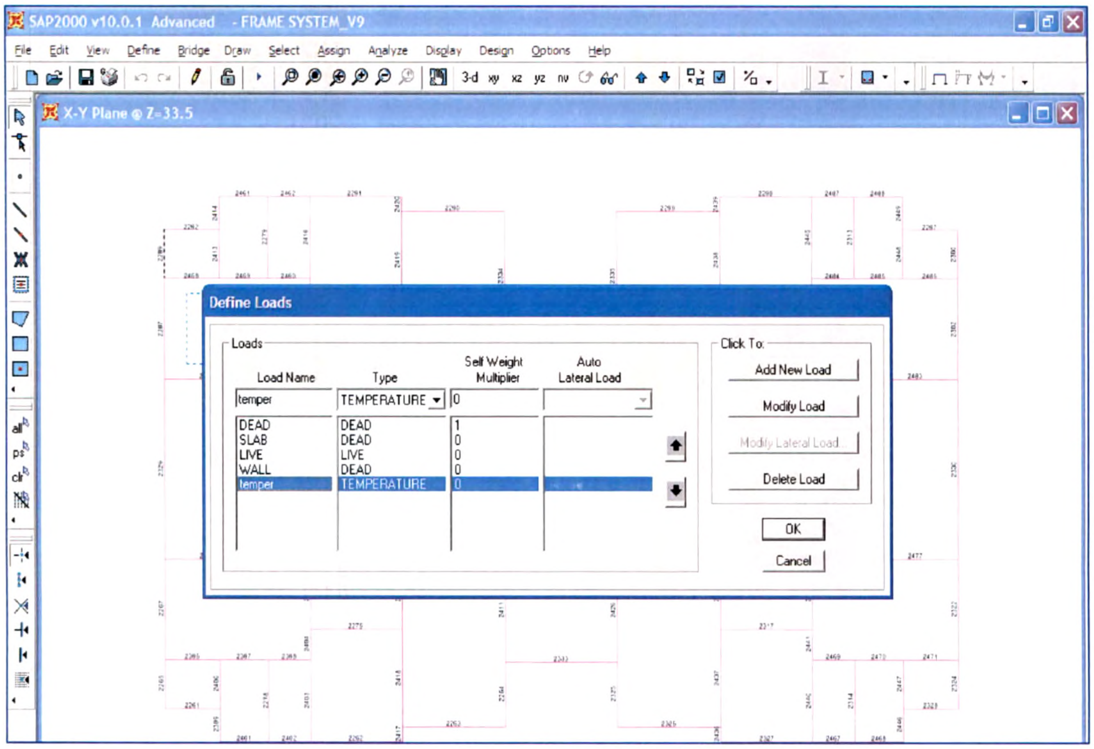

Fig. 8.3 Defining Temperature Load Case in SAP

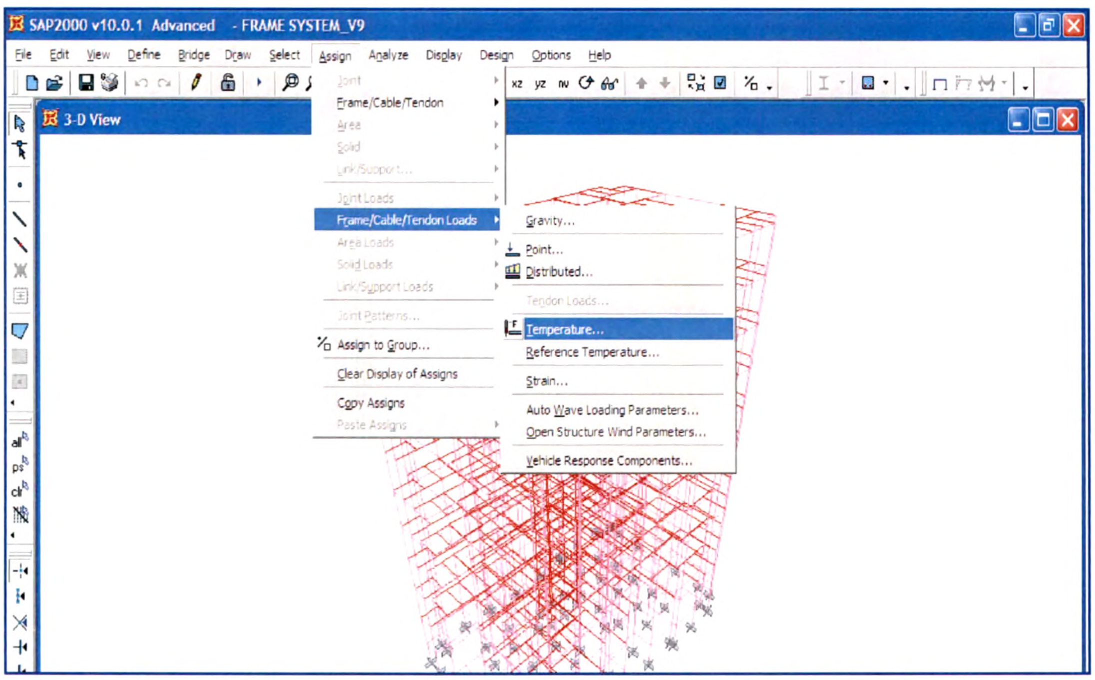

Fig.8.4 Assign Temperature Load to Frame Element Members

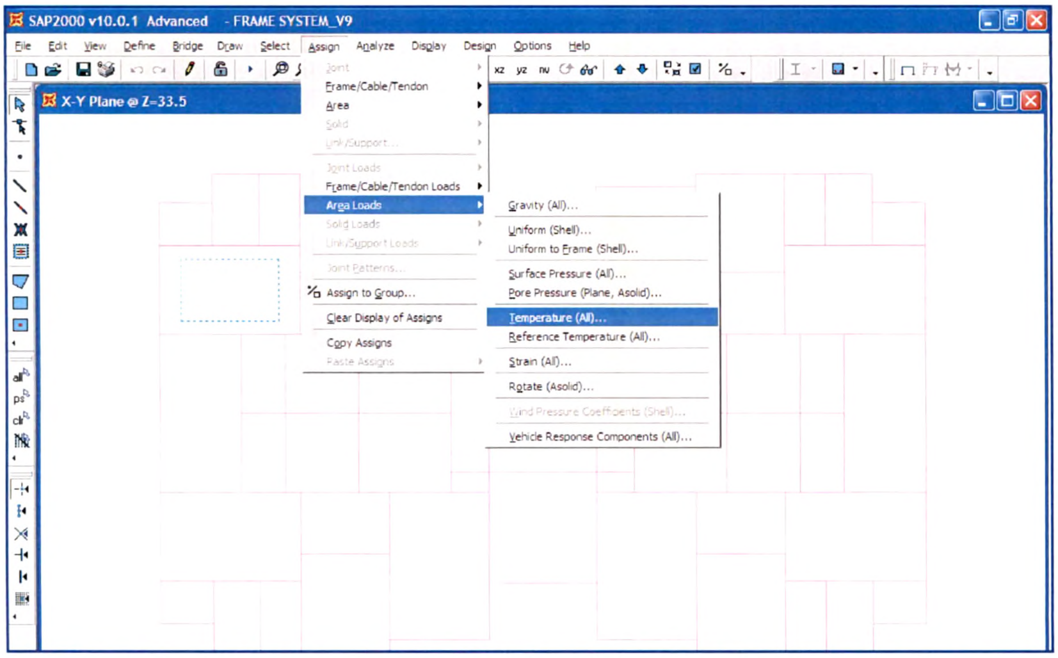

Fig. 8.5 Assign Temperature Load to Slab

8.5.2 Temperature Load on Compartment 1

Compartment 1 is a corner compartment of a top floor (10th slab, 33.5m) as shown in **Fig. 8.6** the member details of this compartment are shown in **Fig.8.7** and **Fig.8.8**

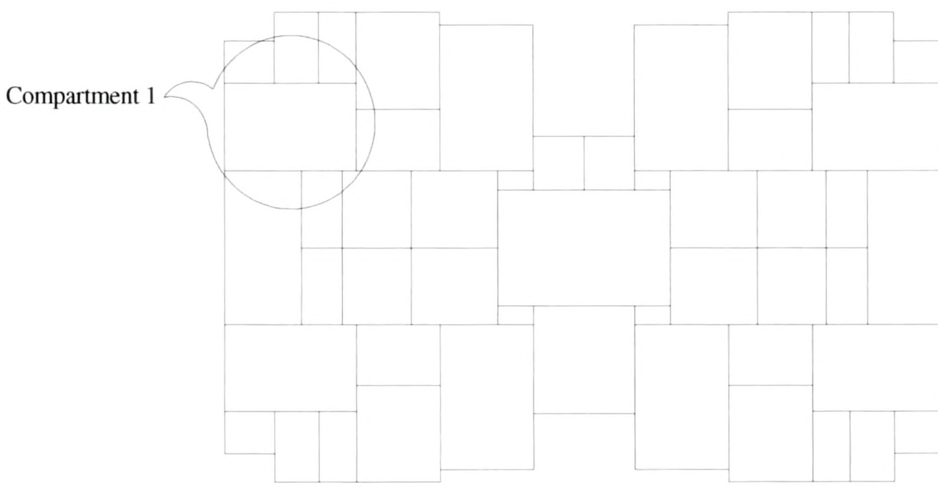

Fig. 8.6 Position of Compartment 1 in Plan of a Building

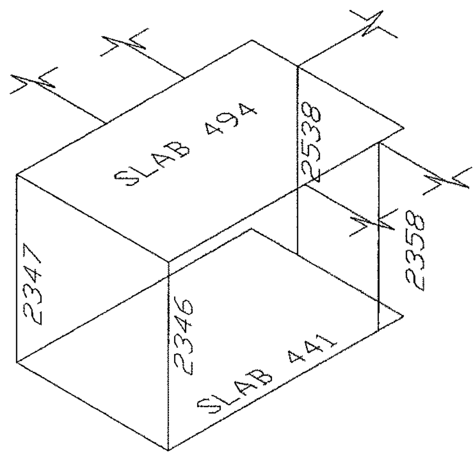

Fig. 8.7 Enlarged View and Columns of Compartment 1

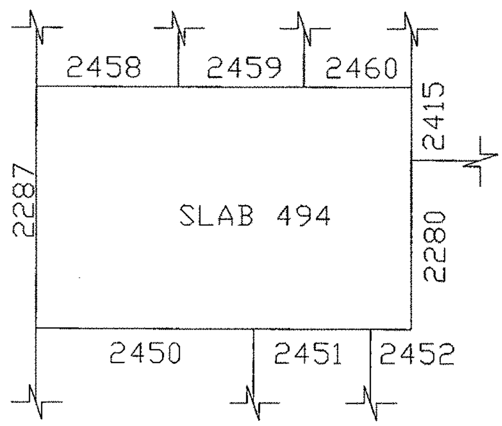

Fig. 8.8 Beams and Slab of Compartment 1 at Top Floor

Generally it is difficult to determine the spread of fire in a building. It is assumed here, that the fire starts from the lower slab and it spreads to the corner columns, and finally engulfs the whole compartment. Hence this compartment is analyzed for a progressive spreading in the compartment. Hence five different Temperature Loads on various elements have been taken as mentioned below:

Case 1: On lower slab 441.

Case 2: On lower slab 441 and corner columns 2347 and 346.

Case 3: On lower slab 441 and all columns.

Case 4: On lower slab 441, all columns and on all beams.

Case 5: On lower slab 441, all columns, all beams and on upper slab 494.

Forces for combination 1.5 (DL+LL) are checked for structural design. After applying the temperature load to different members, the variation in forces due to temperature load and without temperature load are compared. **Table 8.6** shows the member forces for 1.5 (DL+LL) combination. After that the temperature 450°C, 600°C and 1200°C are applied to the members of the compartment for different cases as mentioned above. The highlighted rows show the members to which the temperature load is applied.

Table 8.6 Member Forces of Compartment 1 for (DL+LL)

Member No	Member Type	Axial Force, P(kN)	Torsional Moment, T(kN- m)	Shear Force, V(kN)	Bending Moment M(kNm)	Deflection (m)
2458	Beam	0.00	1.04	3.34	4.70	0.000007
2459	Beam	0.00	0.15	3.81	6.60	0.000011
2460	Beam	0.00	0.53	4.48	6.14	0.000007
2415	Beam	0.00	0.27	22.05	13.49	0.000004
2280	Beam	0.00	0.61	3.82	8.28	0.000024
2450	Beam	0.00	0.13	4.92	8.53	0.000020
2451	Beam	0.00	0.00	1.54	4.01	0.000008
2452	Beam	0.00	1.28	26.55	14.92	0.000003
2287	Beam	0.00	0.03	6.12	4.73	0.000004
2347	Column	24.13	0.00	0.92	7.08	0.000030
2346	Column	9.25	0.00	2.76	8.69	0.000022
2538	Column	50.84	0.00	0.72	2.55	0.000007
2358	Column	39.43	0.00	9.92	16.64	0.000000

8.5.3 Case 1: Temperature Load on Lower Slab 441

Different temperatures are applied to the lower slab 441 of the compartment, and observed that only the forces and the stresses of that particular slab increases; there is no change in forces of beams and columns of the compartment. **Table 8.7** shows the forces and stresses of lower slab 441.We can see that without temperature load maximum force of that slab was 0 kN/m which increases 24764.89 kN/m for 450° C and maximum 67959.47 kN/m for 1200° C. The stresses are increasing from 202.95 kN/m^2 to 543777.9 kN/m^2.

Table 8.7 Effect of Temperature on Slab 441

SLAB 441	Fmax (kN/m)	Smaxtop (kN/m^2)	Smaxbot (kN/m^2)
Comb 1	0	202.95	202.95
450° C	24764.89	198269.7	267332.5
600° C	33403.81	267328.7	267332.5
1200° C	67959.47	543774	543777.9

8.5.4 Case 2 : Lower Slab 441 and Corner Columns 2347 and 2346

Up to 300° C all members are found safe. Members of compartment failed on further increase in temperature. At still higher temperatures members out of compartment also failed.

(a) At 450° C: Table 8.8 and **Fig.8.9** show the effect of 450°C. Forces generated in the members are found to be much higher as compared to forces without temperature effect. Compartment members do not fail up to 450°C. The maximum axial force is found in column 2347, without temperature effect it was 24.134 kN which increases up to 1504.69 kN after applying 450° C temperature. Torsional moments are zero in the columns; only shear forces, axial forces and bending moments of those columns increase due to temperature load. There is no change in the axial force of beams. Torsional moment, shear force and bending moment are higher in beams 2458 and 2450 as compared to other beams, as they are connected to the corner affected columns. Maximum shear force and maximum bending moment observed are 919.856 kN and 903.856 kN-m respectively, for member 2289 while within the compartment they are 332.683kN and 586.532 kN-m. Two corner members, column 2348 and beam 2289 fail.

Table 8.8 Effect of 450°C for Case 2

FORCES IN MEMBERS WITHIN THE COMPARTMENT :							
Member No	Member Type	Axial Force (kN)	Torsion Moment (kN- m)	Shear Force (kN)	B.M. (kN m)	Deflection (m)	Status Fail?
2458	Beam	0.00	25.56	16.80	15.74	0.00001	No
2459	Beam	0.00	4.51	2.25	4.89	0.00001	No
2460	Beam	0.00	10.13	212.97	421.65	0.00133	No
2415	Beam	0.00	11.55	174.59	404.96	0.00075	No
2280	Beam	0.00	1.59	99.05	128.35	0.00003	No
2450	Beam	0.00	0.88	180.20	586.53	0.00367	No
2451	Beam	1504.69	0.00	166.79	465.68	0.00100	No
2452	Beam	96.70	0.00	163.00	439.42	0.00089	No
2287	Beam	46.72	0.00	8.86	21.17	0.00003	No
2347	Column	304.93	0.00	108.47	285.04	0.00055	No
2346	Column	0.00	25.56	16.80	15.74	0.00001	No
2538	Column	0.00	4.51	2.25	4.89	0.00001	No
2358	Column	0.00	10.13	212.97	421.65	0.00133	No
FORCES IN MEMBERS OUTSIDE COMPARTMENT :							
2348	Column	954.66	0.00	26.97	58.50	0.00007	Fails
2289	Beam	0.00	21.37	919.86	903.66	0.00070	Fails

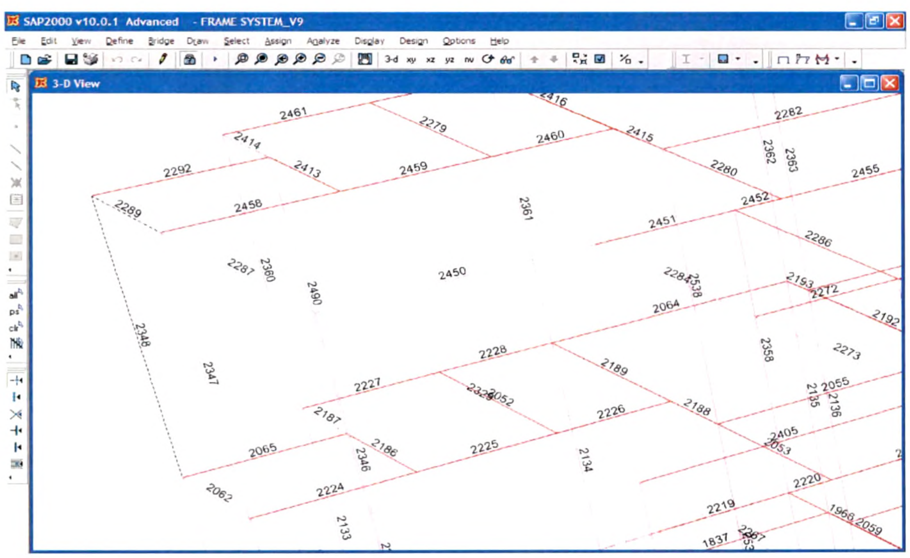

Fig. 8.9 Failure of Corner Members at 450° C Case 2

(b) At **600° C:** Failure of 3 compartment members and 4 members outside the compartment, is observed after applying 600° C temperature to lower slab and corner columns. Member forces in corner columns and members connected to them are higher as compared to other members **(Table 8.9)**. Torsional moments in beams increase significantly while there is no change in torsional moments of columns. Maximum axial force observed for column 2347 is 2037.99kN which was 1504.69kN for 450°C. Maximum shear force and bending moment are 1241.25kN and 1219.76 kN-m for beam 2413 and within the compartment they are 449.899 kN and 790.525 kN-m respectively.

Table 8.9 Effect of 600°C for Case 2

Member No	Member Type	Axial Force (kN)	Torsional Moment (kN- m)	Shear Force, (kN)	Bending Moment, (kN-m)	Deflection (m)	Fails?
FORCES IN MEMBERS WITHIN THE COMPARTMENT :							
2458	Beam	0.00	16.42	449.90	606.71	0.00088	Fails
2459	Beam	0.00	49.86	34.40	140.92	0.00036	No
2460	Beam	0.00	45.92	46.12	63.18	0.00007	No
2415	Beam	0.00	34.38	14.97	16.52	0.00001	No
2280	Beam	0.00	6.30	1.71	3.70	0.00001	No
2450	Beam	0.00	13.61	288.97	571.71	0.00180	No
2451	Beam	0.00	15.58	234.95	546.81	0.00101	No
2452	Beam	0.00	1.70	124.34	167.91	0.00004	No
2287	Beam	0.00	1.18	241.59	790.53	0.00495	Fails
2347	Column	2037.99	0.00	224.65	629.52	0.00136	Fails
2346	Column	133.65	0.00	220.82	595.73	0.00121	No
2538	Column	45.28	0.00	12.20	29.44	0.00005	No
2358	Column	397.55	0.00	149.77	390.27	0.00075	No
FORCES IN MEMBERS OUT OF THE COMPARTMENT							
2348	Column	0.00	34.58	75.53	0.00009	0.00	Fails
2289	Beam	29.09	1241.25	1219.76	0.00094	29.09	Fails
2414	Beam	6.26	572.94	630.44	0.00043	6.26	Fails
2413	Beam	72.84	465.02	697.53	0.00097	72.84	Fails

(c) At **1200° C:** As shown in **Table 8.10,** when temperature is increased up to 1200°C, members of adjacent floors are also affected. Total 14 members fail out of which 4 members are part of compartment and others are out of compartment members. As seen earlier, the axial force is maximum for column 2347 at 2037.99 kN for 600° C and increases up to 4171.23 kN for 1200° C. Maximum torsional moment is 148.97 kNm, maximum shear force and bending moment are 2526.84 kN and 2484.19 kNm for member 2289. Overall for all members shear force, and bending moments are much higher. There is no change in the axial forces in beams, only axial forces of columns increase. Torsional moments are zero in the columns while there is a remarkable change in the torsional moments of beams due to temperature load.

Table 8.10 Effect of 1200° C for Case 2

FORCES IN MEMBERS WITHIN THE COMPARTMENT							
Member No	Member Type	Axial Force, P(kN)	Torsion Moment T(kNm)	Shear Force, V(kN)	Bending Moment M(kNm)	Deflec-tion (m)	Status Fail?
2458	Beam	0.00	34.48	918.76	1239.20	0.00179	Fails
2459	Beam	0.00	101.59	66.04	287.15	0.00074	No
2460	Beam	0.00	92.86	98.46	134.89	0.00016	No
2415	Beam	0.00	69.66	7.65	19.67	0.00002	No
2280	Beam	0.00	13.45	0.48	1.03	0.00000	No
2450	Beam	0.00	27.55	593.00	1171.96	0.00367	Fails
2451	Beam	0.00	31.69	476.41	1114.23	0.00207	No
2452	Beam	0.00	2.12	225.50	326.18	0.00009	No
2287	Beam	0.00	2.36	487.16	1606.50	0.01007	Fails
2347	Column	4171.23	0.00	456.10	1284.89	0.00279	Fails
2346	Column	281.48	0.00	452.11	1220.94	0.00249	No
2538	Column	39.53	0.00	25.57	62.54	0.00011	No
2358	Column	768.02	0.00	314.96	811.21	0.00152	No

Table 8.10 Effect of 1200° C for Case 2 (Contd..)

FORCES IN MEMBERS OUT OF THE COMPARTMENT							
Member No	Member Type	Axial Force, P(kN)	Torsion Moment T(kNm)	Shear Force, V(kN)	Bending Moment M(kNm)	Deflection (m)	Status Fail?
2348	Column	0.00	65.02	143.67	0.00019	0.00	Fails
2490	Column	0.00	54.85	153.32	0.00033	0.00	Fails
2120	Column (9th fl)	0.00	185.91	422.11	0.00059	0.00	Fails
2121	Column (9th fl)	0.00	16.51	31.41	0.00002	0.00	Fails
2289	Beam	59.97	2526.84	2484.19	0.00192	59.97	Fails
2414	Beam	12.80	1153.95	1272.79	0.00087	12.80	Fails
2413	Beam	148.97	937.65	1406.48	0.00196	148.97	Fails
1835	Beam (26.8m)	1.82	730.47	604.20	0.00030	1.82	Fails

8.5.5 Case 3 : Temperature Load Slab 441, and on All Columns

Different temperatures are applied to the lower slab and all columns of the compartment. This is the most critical case, where maximum failure is observed. We can see that when fire affects all the columns of the compartment, it is most devastating.

(a) At 450° C: Four members outside the compartment and one member within the compartment fail at 450° C. Maximum axial force observed is 1485.77 kN for the column 2347. Maximum torsional moment is 65.8609 kN-m for beam 2452. Maximum shear force and bending moment are 905.865 kN and 895.114 kN-m and within the compartment they are 539.889 kN and 810.601 kN-m respectively. For columns; axial forces, shear forces and bending moment increase while there is no change in torsional moments. **Table 8.11** shows the member forces at 450° C for this case.

Table 8.11 Effect of 450° C for Case 3

Member No	Member Type	Axial Force, (kN)	Torsion Moment (kN- m)	Shear Force, (kN)	Bending Moment (kN- m)	Deflection (m)	Status Fails?
FORCES IN MEMBERS WHITHIN THE COMPARTMENT							
2452	Beam	0.00	65.86	539.89	810.60	0.0002	Fails
2287	Beam	0.00	9.02	168.39	574.59	0.0037	No
2347	Column	1485.77	0.00	119.91	337.75	0.0007	No
2346	Column	145.78	0.00	2.86	21.88	0.0001	No
2538	Column	880.42	0.00	230.83	636.18	0.0013	No
2358	Column	0.00	65.86	539.89	810.60	0.0013	No
FORCES IN MEMBERS OF OUT OF THE COMPARTMENT							
2348	Column	943.00	0.00	18.34	38.70	0.0000	Fails
2289	Beam	0.00	15.79	905.87	888.67	0.0007	Fails
2455	Beam	0.00	30.97	732.20	895.11	0.0010	Fails
2413	Beam	0.00	21.99	451.74	677.61	0.0009	Fails

(b) At 600° C: Total thirteen members within the compartment fail in this case, while eight Members of adjacent compartments are also found to have failed. As seen earlier there is no change in axial forces in beams and torsional moments in columns. Shear forces and bending moments increase tremendously. Maximum axial force is 2012.48 kN for the column 2347. Maximum torsional moment is 88.3875 for beam 2452. Maximum shear force and bending moments are 122.38kN and 1209.21 kN-m of the members outside the compartment while the maximum shear force and bending moments of the members within the compartment are 737.482 kN and 1098.57 kN-m respectively. Maximum deflection is observed in beam 2287 which is 0.005019 m. **Table 8.12** shows the member forces for 600° C for this case.

Table 8.12 Effect of 600° C for Case 3

Mem ber No	Member Type	Axial Force, (kN)	Torsion Momen (kNm)	Shear Force, (kN)	Bending Moment (kN m)	Deflection (m)	Status Fails?
FORCES IN MEMBERS WITHIN THE COMPARTMENT							
2458	Beam	0.00	7.83	480.99	474.92	0.0003	No
2459	Beam	0.00	23.57	139.53	366.53	0.0008	No
2460	Beam	0.00	22.49	96.69	132.46	0.0002	No
2415	Beam	0.00	55.58	383.03	634.63	0.0005	Fails
2280	Beam	0.00	39.08	236.86	513.98	0.0015	No
2450	Beam	0.00	22.94	38.30	83.57	0.0003	No
2451	Beam	0.00	9.03	154.30	295.11	0.0005	No
2452	Beam	0.00	88.39	737.48	1098.57	0.0003	Fails
2287	Beam	0.00	12.15	225.67	774.41	0.0050	Fails
2347	Column	2012.48	0.00	161.42	456.96	0.0010	Fails
2346	Column	193.40	0.00	4.82	29.72	0.0001	No
2538	Column	1205.27	0.00	311.60	858.99	0.0018	No
2358	Column	1348.69	0.00	302.92	832.28	0.0017	Fails
FORCES IN MEMBERS OUT OF THE COMPARTMENT							
2348	Column	1276.25	0.00	22.94	48.82	0.0001	Fails
2359	Column	1239.18	0.00	330.34	896.27	0.0018	Fails
2289	Beam	0.00	21.56	1222.38	1199.55	0.0009	Fails
2414	Beam	0.00	5.11	744.36	785.68	0.0005	Fails
2413	Beam	0.00	29.93	606.48	909.72	0.0013	Fails
2282	Beam	0.00	15.10	522.85	799.99	0.0012	Fails
2286	Beam	0.00	15.62	374.57	634.09	0.0001	Fails
2455	Beam	0.00	41.99	988.85	1209.21	0.0014	Fails

(c) At 1200° C:

Maximum failure is observed when all columns of the compartment are subjected to 1200° C temperature. In such circumstances it is observed that these high temperatures not only affect the members of adjacent compartments, but it also affects the members of lower floors. It is observed that, one column of 9th floor and two beams of 8th and 9th slabs respectively fail, and that totally thirty members fail. Maximum axial force observed is 4119.33 kN at 1200° C which is 2012.48 kN for 600° C for column 2347. Maximum shear force, torsional moment, bending moment and deflection are 2488.44 kN, 178.494 kN-m, 2443.07 kN m and 0.010211 m respectively. Bending moments of all members increase remarkably.

8.5.6 Case 5: Temperature Load on Slab 441, All Columns Beams

Maximum failure is observed when temperature Load is given to all columns so maximum failure is found in case 3. When temperature load is applied to columns as well as beams, the extent of member failure remains the same and no further failure is observed. After comparing the tables of case 3 and case 4 it can be inferred that, the member forces are same at 450° C, 600° C, and 1200° C except the axial forces in the beams of the compartment subjected to temperature load. Due to temperature load given to beams of the compartment, their axial forces increase remarkably and for all the beams they are same for same temperature load. The axial forces in the beams observed are 21872.354 kN, 29502.244 kN and 60021.807 kN at 450° C, 600° C, and 1200° C respectively. Here only one **Table 8.13** is attached which shows the effect of 600° C.

Table 8.13 Effect of 600° C for Case 4

FORCES IN MEMBERS OF THE COMPARTMENT :							
Member No	Member Type	Axial Force, (kN)	Torsiona Moment, (kN- m)	Shear Force, (kN)	Bending Moment, (kN- m)	Deflection (m)	Status Fails ?
2458	Beam	0.00	14.87	982.02	971.09	0.0007	No
2459	Beam	0.00	48.10	279.92	746.16	0.0017	No
2460	Beam	0.00	45.21	201.35	275.84	0.0003	No
2415	Beam	0.00	113.35	802.09	1298.41	0.0010	Fails
2280	Beam	0.00	80.15	485.83	1054.24	0.0031	No

Table 8.13 Effect of 600° C for Case 4 (Contd...)

Member No	Member Type	Axial Force,	Torsion Moment	Shear Force,	Bending Moment	Deflectn (m)	Status Fails?
2450	Beam	0.00	46.80	72.82	164.62	0.0006	No
2451	Beam	0.00	18.38	315.53	598.65	0.0010	No
2452	Beam	0.00	178.49	1527.85	2250.47	0.0006	Fails
2287	Beam	0.00	24.69	454.77	1573.72	0.0102	Fails
2347	Column	4119.33	0.00	327.45	933.81	0.0021	Fails
2358	Column	2784.67	0.00	626.55	1710.46	0.0035	Fails
FORCES IN MEMBERS OUT OF THE COMPARTMENT :							
2348	Column	1276.25	0.00	22.94	48.82	0.0001	Fails
2359	Column	1239.18	0.00	330.34	896.27	0.0018	Fails
2289	Beam	0.00	21.56	1222.38	1199.55	0.0009	Fails
2414	Beam	0.00	5.11	744.36	785.68	0.0005	Fails
2413	Beam	0.00	29.93	606.48	909.72	0.0013	Fails
2282	Beam	0.00	15.10	522.85	799.99	0.0012	Fails
2286	Beam	0.00	15.62	374.57	634.09	0.0014	Fails
2455	Beam	0.00	41.99	988.85	1209.21	0.0014	Fails

8.5.7 Comparison of Forces in Critical Members

Forces of the most critical column (2347) and beam (2458) have been studied for their maximum forces and the overall response of the structure.

(a) Column 2347: From **Fig. 8.10 to 8.13**, it can be seen that there is a sudden increase in the forces due to the temperature load as compared to the forces without temperature effect. It is observed that the maximum axial force is in column 2347. Column 2347 is found to resist up to 450°C, after which it fails. Column 2347 is a corner column of the compartment and hence the maximum axial force, shear force, bending moment and deflection are seen in case 2 when the temperature load is applied to lower slab and corner columns as shown in **Table 8.14**

Table 8.14 Comparison of Forces of Column 2347

	Case 1	Case 2			Case 3			Case 4		
		450°	600° (fails)	1200° (fails)	450°	600° (fails)	1200° (fails)	450°	600° (fails)	1200° (fails)
Maximum Axial Force	24.13	1505	2038	4171	1486	2012	4119	1486	2012	4119
Maximum Torsional	0	0	0	0	0	0	0	0	0	0
Maximum Shear	0.92	166.8	225	456.1	120	161.4	327.5	119.9	161.4	327.5
Maximum Bending	7.078	465.7	630	1285	338	457	933.8	337.7	457	933.8
Deflection	3E-05	1E-03	0	0.003	0	0.001	0.002	7E-04	0.001	0.002

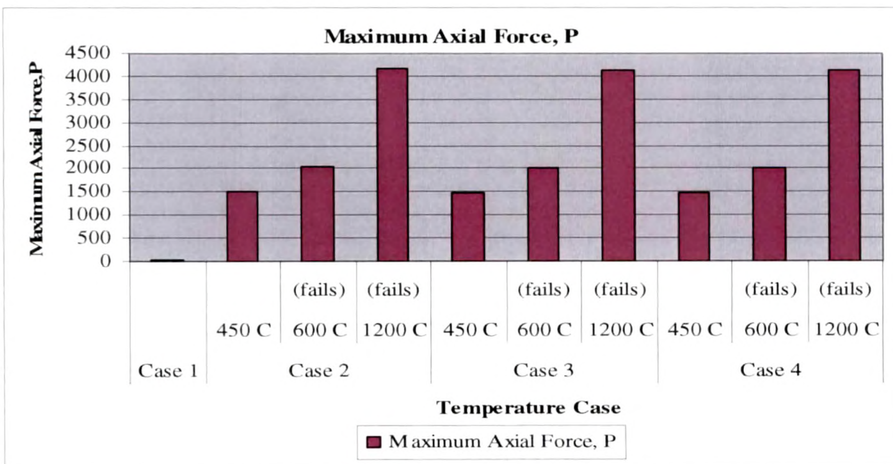

Fig. 8.10 Comparison of Maximum Axial Force for Column 2347

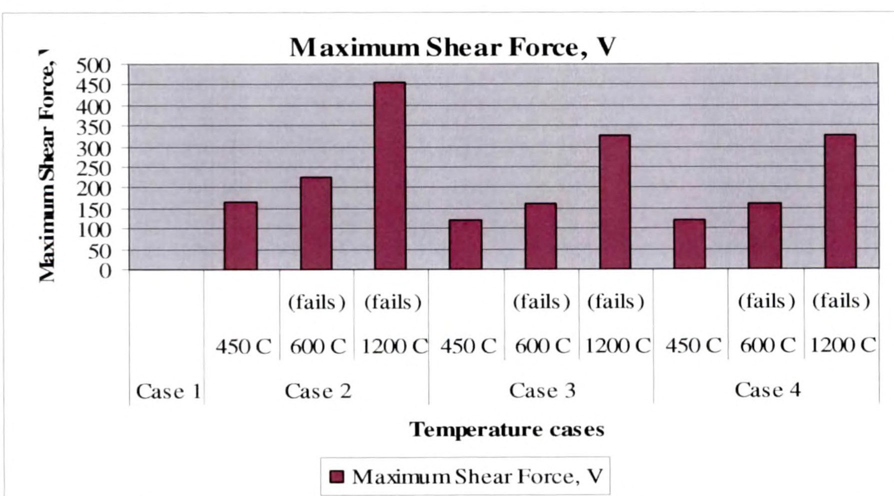

Fig. 8.11 Comparison of Maximum Shear Force for Column 2347

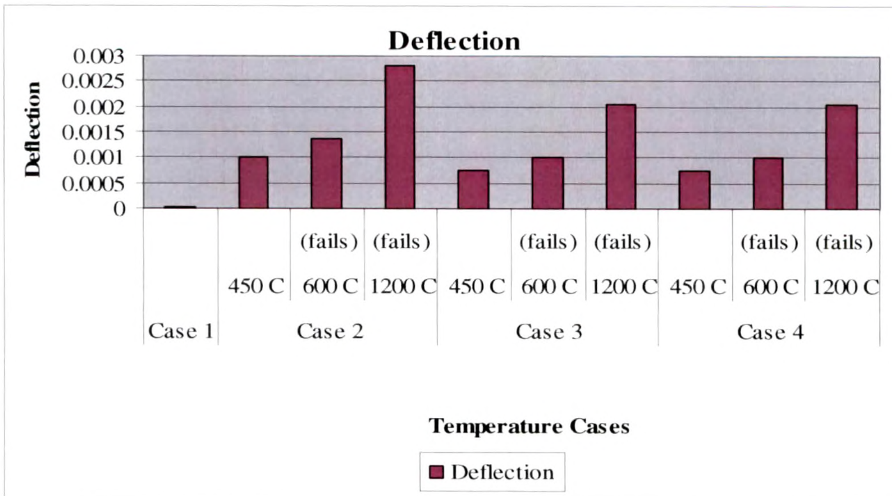

Fig. 8.12 Comparison of Maximum Deflection for Column 2347

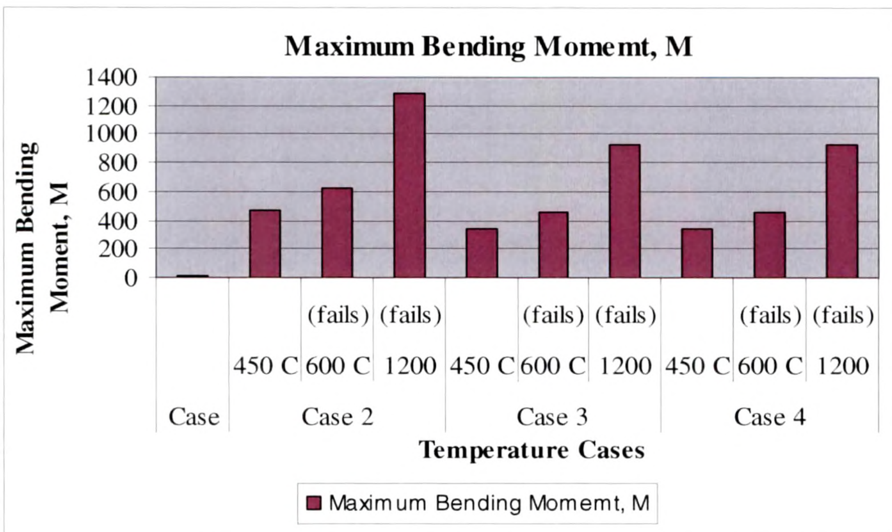

Fig. 8.13 Comparison of Maximum Bending Moment for Column 2347

(b) Beam 2458: Beam 2458 is connected to corner column, hence beam 2458 too, the forces are maximum for case 2 when temperature load is applied to lower slab and corner columns. (**Table 15; Figs. 8.14 - 8.18**). Another interesting result is that when temperature load is subjected on corner columns only than the beam fails at 600° C while when the temperature load is given to all the columns simultaneously, it can resist up to 600° C and it fails at 1200° C. There is a remarkable difference in the torsional moments and deflection for case 2 than other cases. Thus it is clear that the beam is most affected when the temperature load is given to the column connected to it.

Table 8.15 Comparison of Forces of Beam 2458

	Case 2			Case 3			Case 4		
	450°	600° (fails)	1200° (fails)	450°	600°	1200° (fails)	450°	600°	1200° (fails)
Maximum Axial Force	0	0	0	0	0	0	21872	29502	60022
Maximum Tortional	11.90	16.42	34.47	6.07	7.834	14.86	6.077	7.835	14.87
Maximum Shear	332.6	449.9	918.7	355.7	480.9	982.0	355.7	481	982
Maximum Bending	448.5	606.7	1239.	350.8	474.9	971.0	350.9	474.9	971.1

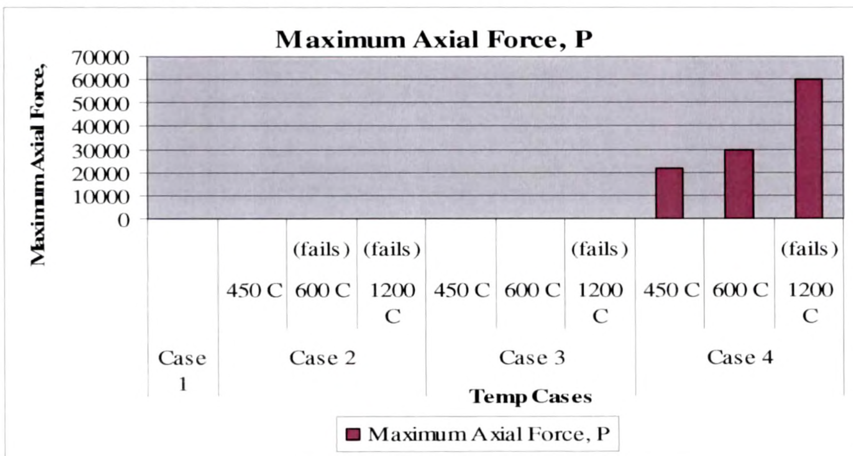

Fig. 8.14 Maximum Axial Force for Beam 2458

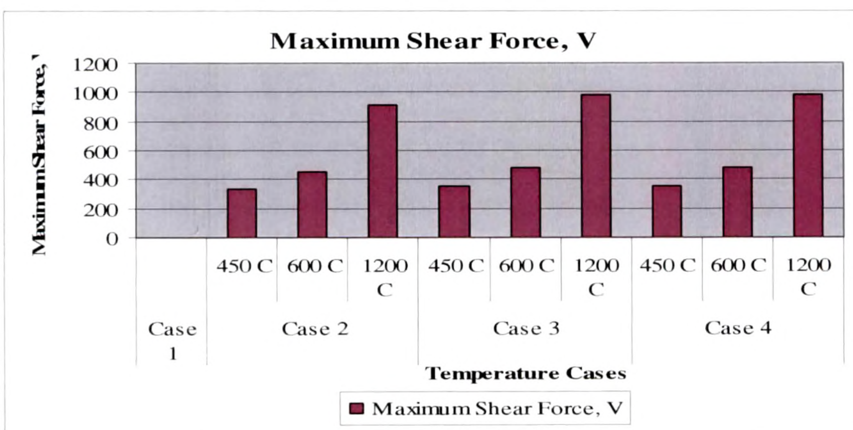

Fig. 8.15 Maximum Shear Force for Beam 2458

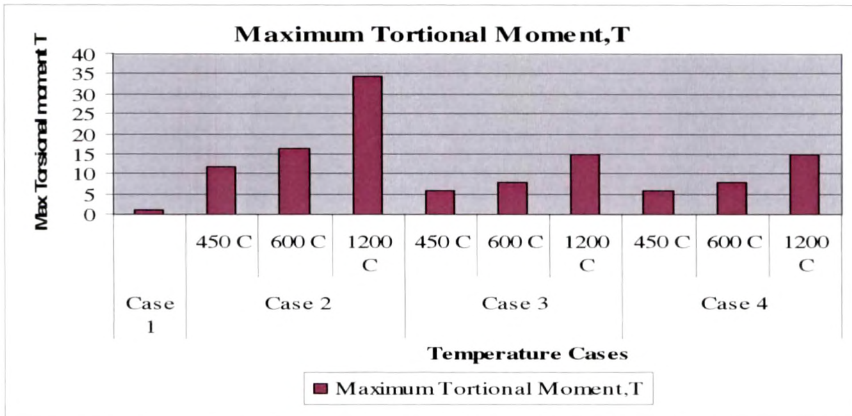

Fig. 8.16 Maximum Torsional Moment for Beam 2458

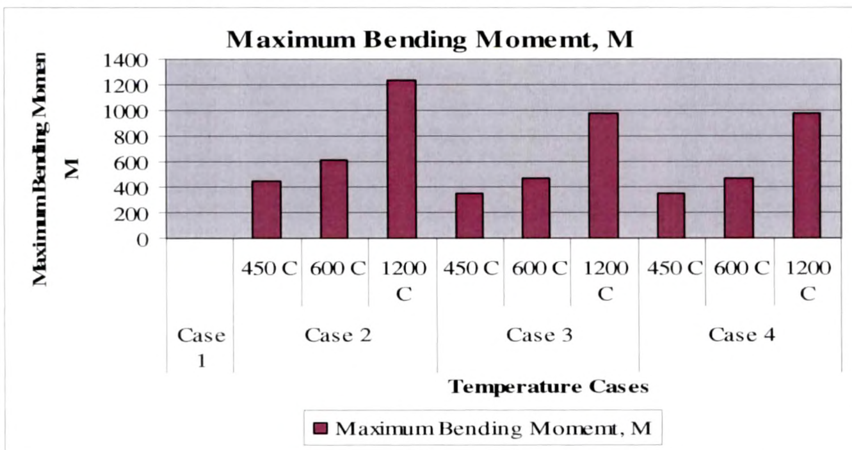

Fig. 8.17 Mximum Bending Moment for Beam 2458

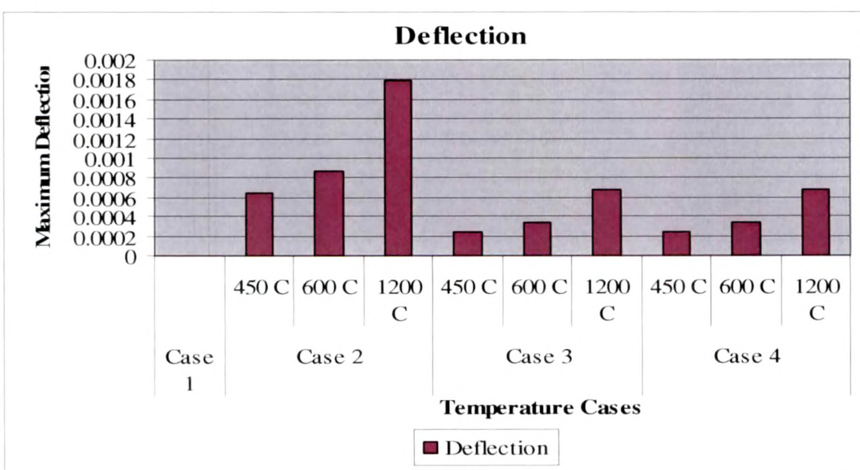

Fig. 8.18 Maximum Deflection for Beam 2458

8.5.8 Temperature Load On Compartment 2

Compartment 2 is an interior compartment of a top floor as shown in **Fig. 8.19**. The member details of this compartment are shown in **Fig.8.20 and Fig.8.21**. Up to 300°C all members pass the design checks, so here member forces at 450°C, 600°C and 1200°C are checked.

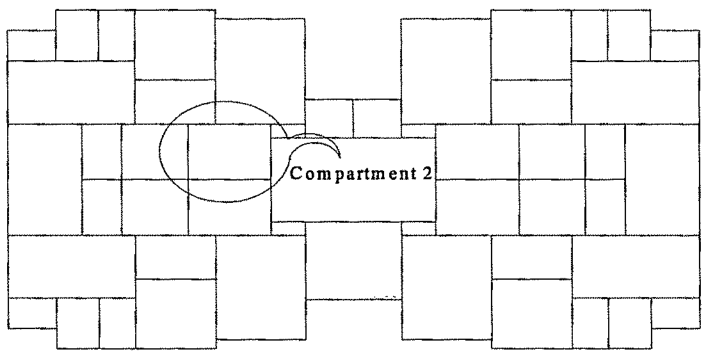

Fig. 8.19 Position of Compartment 2 in Plan

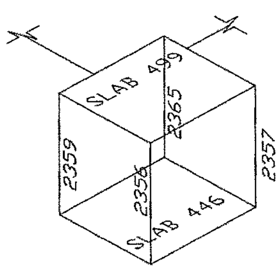

Fig. 8.20 Columns

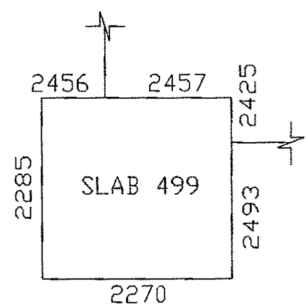

Fig. 8.21 Beams and Slab

Compartment 2 is also checked for five cases as mentioned for compartment 1.

Case 1: Temperature load on lower slab 446.

Case 2: Temperature load on lower slab 446, corner columns 2356 and 2359.

Case 3: Temperature load on lower slab 446, and all columns.

Case 4: Temperature load on lower slab 446,all columns and all upper beams.

Case 5: Temperature load on lower slab 446, all columns, all upper beams and on upper slab 494.

Each load case comprises an increase in temperature from 450°C to 1200°C. The building shows progressive behaviour with its members failing as the fire temperatures keep increasing and more and more members are exposed to the fire loads. But a few critical beams and columns, due to their positioning and continuity or discontinuity with other members inside the structures invariably fail for most load cases. These are mostly Beams and Columns which are on the corners and show a highly vulnerable disposition to fire loads. **Table 8.16** shows forces in such vulnerable members for all the load cases and all the temperature gradients.

Table 8.16 Critical Members for All Load Cases.

FORCES IN CRITICAL BEAMS AND COLUMNS							
Memb er No	Member Type	Axial Force (kN)	Torsio nal (kN m)	Shear Force (kN)	Bending Moment, (kN- m)	Case	Status Fails?
2456	Beam	0.00	1.70	19.59	10.97	**DL+LL**	No
2493	Beam	0.00	0.08	9.83	15.13		No
2356	Column	52.92	0.00	5.34	10.65		No
2365	Column	20.53	0.00	0.55	1.29		No
					Case2	**450°**	
2456	Beam	0.00	31.53	712.97	986.78		Fails
					Outside	***Comp***	
2452	Beam	0.00	33.24	788.06	862.75		Fails
2455	Beam	0.00	1.69	820.16	1110.57		Fails
2355	Column	1221.05	0.00	364.58	1036.16		Fails
					Case 2	**600°**	
2456	Beam	0.00	41.94	968.51	1334.84		Fails
2493	Beam	0.00	13.46	0.15	10.14		No
2425	Beam	0.00	11.97	41.78	34.92		No
2356	Column	2361.99	0.00	86.69	252.58		No
2365	Column	893.62	0.00	249.20	663.46		No
					Outside	***Comp***	
2452	Beam	0.00	44.39	1053.71	1158.51		Fails
2455	Beam	0.00	2.07	1105.04	1496.13		Fails
2355	Column	1221.05	0.00	364.58	1036.16		Fails
					Case2	**1200°**	
2456	Beam	0.00	83.56	1990.69	2727.04		Fails
2493	Beam	0.00	27.30	10.47	36.29		No
2356	Column	4860.17	0.00	170.84	502.85		Fails
2365	Column	1796.82	0.00	506.43	1348.46		Fails

Member No	Member Type	Axial Force (kN)	Torsional (kN m)	Shear Force (kN)	Bending Moment, (kN- m)		Case
					Outside Comp		
2452	Beam	0.00	88.99	2116.30	2341.53		Fails
2455	Beam	0.00	3.56	2244.56	3038.36		Fails
2283	Beam	0.00	1.19	470.68	614.24		Fails
2355	Column	2431.48	0.00	740.97	2108.22		Fails
2354	Column	860.38	0.00	16.21	35.17		Fails
					Case3 600°		
2456	Beam	0.00	60.15	276.18	420.54		No
2493	Beam	0.00	88.86	609.72	813.83		Fails
2356	Column	1675.64	0.00	310.41	858.35		Fails
2365	Column	2515.65	0.00	548.21	1513.24		Fails
					Outside Comp		
2452	Beam	0.00	50.36	930.20	1122.53		Fails
2455	Beam	0.00	2.19	976.91	1217.94		Fails
2355	Column	1098.39	0.00	361.68	1005.23		Fails
2358	Column	1059.45	0.00	353.22	957.30		Fails
					Case3 1200°		
2456	Beam	0.00	120.61	582.15	866.93		Fails
2493	Beam	0.00	180.85	1250.64	1671.37		Fails
2356	Column	3463.80	0.00	625.99	1735.27		Fails
2365	Column	5139.29	0.00	1114.75	3077.33		Fails
					Outside Comp		
2452	Beam	0.00	101.14	1865.02	2268.32		Fails
2455	Beam	0.00	3.81	1983.87	1485.43		Fails
2283	Beam	0.00	27.62	2985.57	3896.17		Fails
2355	Column	2181.94	0.00	735.08	2045.28		Fails
2358	Column	2114.64	0.00	708.36	1930.39		Fails
					Case 4 600°		
2456	Beam	29502.24	60.15	276.18	420.54		No
2493	Beam	29502.24	88.86	609.72	813.83		Fails
2356	Column	1675.64	0.00	310.41	858.35		Fails
2365	Column	2515.65	0.00	548.21	1513.24		Fails
					Outside Comp		
2452	Beam	0.00	50.36	930.20	1122.53		Fails
2455	Beam	0.00	2.19	976.91	1217.94		Fails
2283	Beam	0.00	13.79	1467.00	1914.43		Fails
2355	Column	1098.39	0.00	361.68	1005.23		Fails
2358	Column	1059.45	0.00	353.22	957.30		Fails

8.5.9 Case 3: Compartment 2

(a) At 600° C: In this case totally fourteen members are found to fail. Maximum axial force is observed in column 2365, which increases from 1859.744kN to 2515.653 kN from 450° C to 600° C. There is no change in axial forces of beams and torsional moments of columns. Shear forces and bending moments of all members increase tremendously. The maximum shear force and bending moment are 1467 kN and 1914.429 kN-m for beam 2283. Maximum torsional moment and deflection are 121.1069 kN-m for beam 2425 and 0.003188 m for column 2365 respectively.

(b) At 1200° C: As seen for compartment 1, even in compartment 2, maximum failure is observed when 1200° C temperature is applied to all columns of the compartment. It is observed that, it not only affects the members of adjacent compartments but it also affects the members of lower floors. Nine out of ten compartment members fail. One column of 9th floor, three beams of 9th slab and one beam of 8th slab fail. Totally thirty three members fail. Maximum axial force observed is 5139.293 kN for column 2365. Maximum torsional moment, shear force, bending moment and deflection are 247.1446 kN m, 2985.57 kN, 3896.1743 kN m and 0.006485 m respectively.

8.5.10 Case 4: Lower Slab 446, All Columns and on All Beams.

When temperatures 450° C, 600° C, and 1200° C are applied to the members of the compartment, except the axial forces in the beams all the other forces of all the members remain same for the case 3 and case 4. Due to temperature load given to beams of the compartment, their axial forces increase remarkably and for all the beams they are same for same temperature load. The axial forces in the beams observed are 21872.35 kN for450° C, 29502.244 kN for 600° C and 60021.807 kN for 1200° C respectively. Thus the force increases almost 3 times.

8.5.11 Comparison Of Forces In Critical Members Of Compartment 2

(a) Column 2365: Forces in column 2365 increase tremendously due to temperature load as seen in **Table 8.17**. For case 2 even though the temperature forces are not applied to this column (they are applied to corner columns), the forces are much higher as compared to the case 1 due to increase in the forces of the members connected to it. When the temperature load is applied only to the corner columns this column can resist up to 600° C, but fails at 1200° C; But when the temperature load is applied to all the columns it fails at 450° C. Thus this column is a critical member which will initiate progressive failure in the building.

Table 8.17 Comparison of Forces of Column 2365

	Case 2		Case 3			Case 4		
	600° C	1200° C (Fails)	450° C (Fails)	600° C (Fails)	1200° C (Fails)	450° C (Fails)	600° C (Fails)	1200° C (Fails)
Max P (kN)	893.62	1796.82	1859.74	2515.65	5139.29	1859.74	2515.70	5139.30
Max T (kN m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Max V (kN)	249.20	506.43	406.57	548.21	1114.75	406.57	548.21	1114.80
Max M (kN m)	663.46	1348.46	1122.22	1513.24	3077.33	1122.22	1513.20	3077.30
Def. (m)	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.01

(b) Beam 2456: From above figures, for beam 2456 of the compartment 2, we can say that the forces are much higher due to temperature load as compared to the forces without temperature load. We can see that the axial force increases only when the temperature load is given to that beam for all the other cases it is zero. The torsional moment increases with the increase in the temperature load and it is maximum when the temperature load is given to all columns. It is found that because the beam is connected to the corner column, shear force, bending moment and deflection is much higher when the load is given to the corner columns than any other cases as shown in **Table 8.18**.

Table 8.18 Comparison of Forces of Beam 2456

	Case 2			Case 3			Case 4		
	450° C (Fails)	600° C (Fails)	1200° C (Fails)	450° C	600° C	1200° C (Fails)	450° C	600° C	1200° C (Fails)
Maximum P (kN)	0	0	0	0	0	0	21872	29502	60022
Maximum T (kN m)	31.532	41.938	83.562	45.03	60.15	120.6	45.03	60.15	120.61
Maximum V (kN)	712.97	968.51	1990.7	199.7	276.2	582.2	199.7	276.2	582.15
Maximum M (kN m)	986.78	1334.8	2727	308.9	420.5	866.9	308.9	420.5	866.93
Deflection (m)	0.0008	0.0011	0.0023	3E-04	4E-04	8E-04	3E-04	4E-04	0.0008

8.6 FIRE SAFE DESIGNS: MITIGATION ASPECTS

In many countries, it is a statutory requirement to design all structures to withstand fire for a specified period. The trend is shifting from prescriptive codes to performance based criteria as mentioned earlier. Thus it now becomes imperative to provide adequate structural failure criteria as part of the overall fire safety sysetrm. As a necessary basis for fire behaviour, the response of the structure and its members, to the influence of all important parameters under ambient conditions are first examined. This is followed by an assessment of the fire loads and finally the highlights of structural fire design are examined. Flow chart in **Fig. 8.22** outlines the procedure for fire safety design.

8.7 FIRE RESISTANCE RATING

Another criteria that most codes prescribe is the FRR. The objective is to save lives by preventing the spread of fire and to ensure that the structure does not collapse before it has been safely evacuated. The fire resistance rating is defined as the time elapsed during a fire up to the time the structure loses its load-bearing or protective capacity. Though fires still constitute a major hazard, most concrete structures remain serviceable even after a fire, provided the damaged areas are expertly repaired.

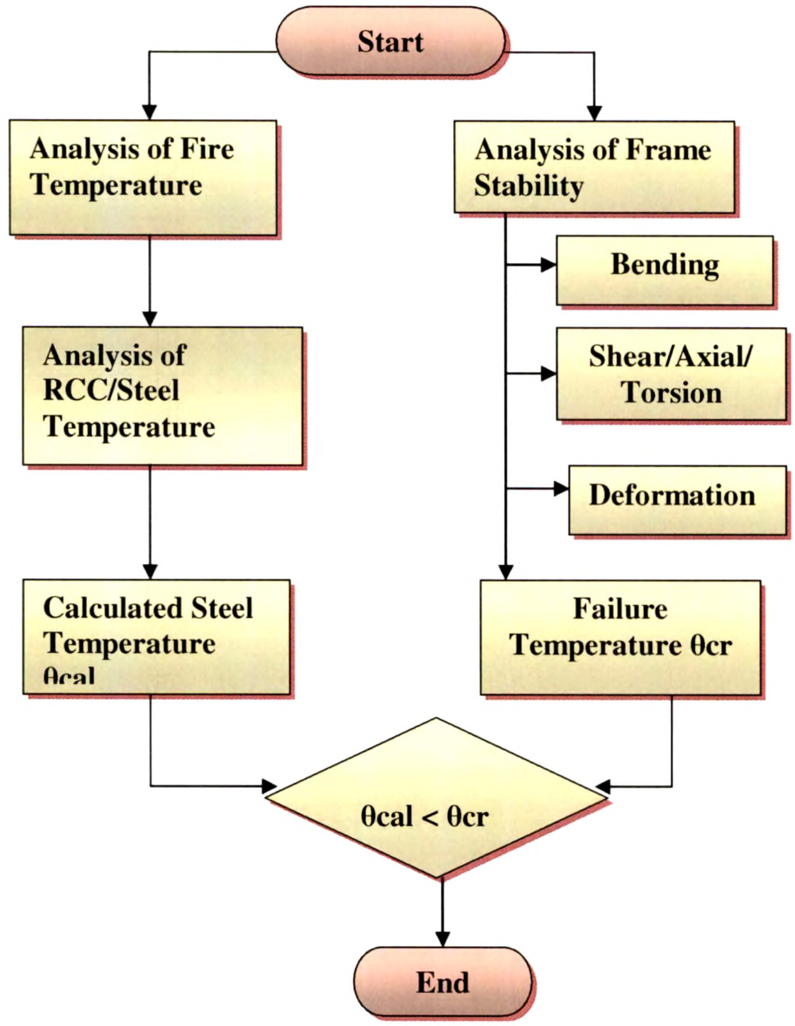

Fig. 8.22 Flow of Fire Safety Design for Framed Structure

8.7.1 Fire Resistance of Reinforced Concrete Structures

Fire resistance of reinforced concrete structures depends on many design decisions and the way they are implemented during the construction of buildings. The fire resistance of axially compressed columns with flexible reinforcement bars depends upon the cross section of the column, the thermal properties of the material, the coefficient of change in strength of concrete under high temperatures, and the corresponding critical temperature. Therefore, to increase the fire resistance of columns, the cross section of the column should be increased or lower density concrete should be used that has a lower thermal conductivity or the load on the column should be decreased. Compressed elements with highly eccentric loading fail when tensile stressed reinforcement bars heat up, the fire resistance of such elements may be increased by protecting these reinforcement bars from high temperature. This can be done by increasing the thickness of cover to the reinforcing bars, or by applying a proper coat of plaster.

The fire resistance of freely-supported slabs and beams can be increased by increasing the thickness of cover of concrete and decreasing the coefficient of thermal conductivity of the concrete, by applying proper plaster with low heat-conducting materials, decreasing the load, and using reinforcement bars with higher critical temperatures. Thus it is known that, low alloy reinforcement steel is better than high-strength, cold-drawn reinforcement steel or hot-rolled reinforcement bars [103]

Fire can cause 'cracking' and 'spalling' of concrete. Cracking may occur in certain areas, such as in the negative moment regions of continuous beams and slabs. It is therefore essential, at the design stage itself, to provide adequate concrete cover over the reinforcing steel (As per IS: 456-2000) in order to prevent the flames from heating it. The mechanism of spalling is rather complex. Limestone aggregate is generally superior to siliceous aggregate in resisting spalling; lightweight aggregates also provide improved fire resistance to the concrete. Secondary Reinforcement in the form of wire mesh should be used by placing them in the midway within the depth of the concrete cover, in places where spalling is likely to be serious problem.

8.8 DESIGN OF FIRE WALLS

The effectiveness of fire walls derives from their non-combustibility, adequate heat resistance and stability, and adequate density and impermeability to block the spread of combustion products during fire. According to existing standards at least a four-hour fire resistance rating is required for fire walls that divide buildings into fire areas or sections, and at least two hours for the walls within these sections. These conditions are easily satisfied if the main walls used for this purpose are made from small units such as bricks or blocks. A wall 25 cm thick made from ordinary bricks has a fire resistance rating of 5.5 hours. In practice, brick fire walls are at least 38cm thick, and this satisfies the requirements of the fire safety standards. Walls of concrete blocks also have sufficient thickness and hence a sufficiently high fire resistance rating. Walls made of natural stones also withstand well during fire.

As we know, the fire resistance limit of reinforced concrete columns depends upon their cross sectional area and the manner in which they are loaded. The fire resistance limit of reinforced concrete elements subjected to bending stresses is less than that of elements subjected to axial compression. This means that reinforced concrete columns subjected to highly eccentric off-center compression will have a fire resistance limit lower than specified. Therefore fire walls should have columns that are axially compressed. When frame-and-panel type design is adopted for a fire wall; the beam is subjected to heating from three sides. According to the standards, the fire resistance limit of such a wall does not exceed 1.5hrs. [103].

Correct execution of joints in fire walls is important for the effectiveness of these walls during fire. The joining of these components involves attaching the wall panels to the framework, butt joining of panels with one another, and joining the roof and ceiling structures. When the panels attached to the columns by means of metallic fasteners, the fire resistance limit of the wall is determined from the fire resistance limit of the joining component, which does not exceed 0.25 hr. If the metallic fastening components are covered with a 3 cm layer of concrete, the fire resistance rating can be increased to 1 hr. If the thickness of the concrete layer is 5 cm, the fire resistance rating is 2hrs. This show in order to have a 4 hrs fire resistance rating, these fastening components should be protected with a layer of concrete not les than 8 cm thick.

Additional stresses created in a fire wall exposed to one-sided destruction should be taken into account. When a structure is embedded in a fire wall from both sides, there is a possibility of the fire passing from one component into another.

In this case, a partition 12 cm thick is placed between the ends of the two beams. In modern frame structure type buildings, the major brick walls are located at expansion joints. During designing of fire walls we should also calculate the stability of wall against tilting.

8.9 FIRE SAFETY ASPECTS GIVEN IN INDIAN CODES

IS codes give [104] detailed information about smoke and fire venting, ducts, air-conditioning, fire lifts, floor area ratio, fire escapes, electrical services etc. some of the important points are mentioned below.

According to IS 1642: 1989 [105], Area more than 750m^2 on individual floor should be segregated by a fire wall and automatic fire dampers for isolation. When openings in floors are provided to allow cables, the space should be protected properly. In case of openings provided to allow plumbing/gas/steam pipes and similar services should be sealed with filler material of fire rating not less than 1 hour. Air-conditioning systems should be so installed and maintained as to minimize the danger of spread of fire. Escape routes like staircases, common corridors, lift lobbies, etc, should not be used as return air passage. Provision has to be made for venting which allows escape of hot gases and smoke released by accidental burning of combustible material stored or are being processed inside a building, and will give ample time to escape either in part or wholly in the event of fire. Service ducts should be enclosed by walls and doors of 2 hours fire rating.

Each basement should be separately ventilated. Vents with c/s area not less than 2.5% of the floor area spread evenly round the perimeter of the basement should be provided. The staircase of basement should be of enclosed type having fire resistance of not less than 2 hours. Ground and upper storey should communicate with basement through a lobby provided with fire resisting self-closing doors of 1 hour fire resistance. Fire lifts should be provided with a minimum capacity for 8 passengers. The electric distribution should be laid in a separate duct. An independent and well-ventilated service room should be provided on the ground floor with direct access from outside or from the corridor.

As per IS: 1644 (1988), All the buildings which are more than 15m in height and all buildings having area more than 500 m^2 on each floor should have a minimum two staircases. Number of exits for educational, assembly, institutional, industrial, storage, and hazardous occupancies should be provided as per this code. Every exit doorway should open into an enclosed stairway, or horizontal exit of a corridor, or passageway providing continuous and protected means of egress. Interior stairs should be constructed with non-combustible materials. No gas piping should be laid in the stairway. Minimum width of tread should be 25cm for residential and 30 cm for other buildings. Fire escape stairs should have straight flight not less than 75 cm wide with 20 cm treads and risers not more than 19cm. The number of risers should be limited to 15 per flight. No staircase, used as a fire escape, should be inclined at an angle greater than 45° to the horizontal. The slope of a ramp should not exceed 1 in 10. In certain cases, steeper slopes may be permitted but no case greater than 1 in 8. IS: 1643 -1988 gives information about the floor area ratio and open spaces for residential and high rise buildings.

8.10 RETROFITTING FOR DAMAGE BY FIRE

The first requirement after a fire, is to decide which concrete members have been so badly damaged that they need to be dismantled and which members can be retrofitted. For that some tests like;' Ultrasonic Pulse Velocity test', 'Schmidt hammer Test', 'Ball Impact test 'etc. should be done. These tests will indicate the extent to which the concrete has suffered a loss in strength and, in some cases, in reinforcing steel.

If the assessment of a structure after fire shows minor damage, the approach to reinstatement work could be on the following lines: surface blackened by smoke and soot are first sandblasted lightly; minor spalls are then treated by applying a suitable mortar mix, bonding agent and cement wash; the more deeply spalled areas are repaired by providing fine welded-wire mesh, latex bonding agent and mortar in thin layers until the specified profile is attained. If necessary, a cement wash can also be applied to obtain a uniform finish on the concrete surface.

Most fires cause only superficial damage to concrete and in such case, it is possible to replace with equivalent material of concrete or steel which is lost or weakened. But, in cases of severe fire damage, the "equivalent replacement" approach is not sufficient and detailed structural calculations are necessary to assess the additional concrete and steel that are required. Where such additional rebars have to be placed, it is advisable to provide new links and/or stirrups having appropriate anchorage lengths and bond characteristics. For this purpose, suitable fixings, bolt anchors and welding provide the means for mechanical anchorage generally required by the links, stirrups and wire fabric reinforcement.

Whatever is the method of repair adopted, the first step would be to remove all deteriorated or loose concrete until only sound material is exposed so as to provide a good bond. To replace the spalled concrete, three methods are commonly used, namely, hand-applied mortar, concrete cast in formwork, and gunite. The first method is labour-intensive and require great care in surface preparation and supervision, if it is therefore, adopted mainly in situations where fire is confined to a relatively small area. As hand-applied mortar is not likely to develop adequate compressive strength, it is used primarily for the restoration of cover to rebars or for cosmic purposes; it is placed in layers up to a thickness of 30mm. The second method is especially economical and effective where the thickness of concrete is adequate to permit placement and compaction. It is also preferred where columns are to be sleeved to provide an increased section

The usual method of repairing major fire damaged concrete structures is guniting. It can be adapted to extremely thin sections or to form layers in any desired shape without the need for formwork to retain the concrete. In "dry" guniting, only the cement and aggregate are mixed and delivered by compressed air through a hose to a nozzle, where water is added; in the "wet" method the concrete mix containing water is fed to the nozzle pneumatically or by pumping. Any cracks in the concrete should also be grouted with suitable epoxy resin compounds applied under high pressure. A further improvement in bonding the gunite can be achieved by providing mechanically fixed supplementary mesh reinforcement in the repair work. Depending on local factors and the direction of spraying, the thickness of the individual layers of gunite generally range from 20mm to 50mm, because adverse effects during setting, such as shrinkage, vibrations or low temperatures can be detrimental to thin gunited sections, the repaired surfaces should be properly cured and, if necessary, be protected from cold weather and dynamic loads during this period.

After a severe fire, selective replacement of some members may be necessary, duly considering its effect on the remaining members; for example, removal of one span of a continuous beam will affect stresses in the remaining spans or may increase the unrestrained length of a column. Joints between the old and new members will also need special consideration. Sometimes, it is possible to provide alternative supports in the form of new beams and columns to sub divide floor spans, and such schemes may well prove economical.

8.11 CLOSURE

Study of corner compartments versus inner compartments shows that the forces are higher for same temperature loads in members of the inner compartment. This may be attributed to the stiffness of the element due to the continuity of elements in all 3 directions. Further torsion which is neglected for any ordinary frame under static loads plays a critical role in initiating failure of members. A typical failure pattern that emerges out of the various load cases, is that of twisting of the compartment as a whole, thus in the process the connected beams and columns undergo large amounts of torsion and bending which they have not been designed for. Thus the present codes which mention only the cover to steel in RC members as a protection to fire, do not suffice in dealing with the fire scenario as a whole.

Fire safe designs as prescribed by various international codes and confirming to performance based fire design, should be especially implemented, as most fire damaged structures can be safely rehabilitated if the original design has included fire aspects. Besides this fire safety norms and by laws for egress and exit, if implemented, can save many lives and do not cost too much extra compared to their effectiveness and the entire cost of the structure.