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Study Steel Reinforced Concrete Beam Column Joints Performance under Exposure to Fire

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Abstract: Fire safety or fire resistance is a critical design consideration for high rise buildings since fire represents one of the most severe conditions that may be encountered during the life time of a building. Beam column joints are main structural components of a building. This study aims to investigate the behaviour of steel-reinforced beam column joints under combined loading and fire including heating cooling and post fire phases. Different I sections are using as structural steel sections for the study. A finite element analysis model was developed to stimulate the response of SRC beam column joints in whole loading sequences, including initial loading ,heating cooling and post fire loading. Also aims to study the relationship between sectional properties of structural steel section and fire resistance behaviour. Comparison of fire resistance behaviour of beam column joints under initial heating phase, cooling phase and post fire loading phase is also discussed.
Keywords: : Fire safetySRC Beam column joints; initial heating loading ;cooling phase loading ; post fire loading phase.

I. INTRODUCTION

Steel reinforced concrete (SRC) is a type of steel- concrete composite structure, which combines the advantages of both steel and concrete, and thus has favorable structural performance. fire resistance (structural fire safety) is a critical design consideration in high rise buildings, since fire represents one of the most severe conditions that may be encountered during the life time of a building.The SRC structure is composed of shaped steel, reinforcing bars, stirrups, and concrete. Compared to more commonly studied concrete filled steel tube (CFST) structures, this type of construction obtains even better fire resistance since the fire vulnerable steel sections are protected by encasing concrete.. Fire resistance is defined as the duration during which a structural member exhibits resistance with respect to structural integrity, stability, and temperature transmission under fire conditions.

II. BEAM COLUMN JOINT WITH DIFFERENT I SECTION STEELS

To study about the fire performance of steel reinforced beam column joints, beam column joints different I sections as structural steel sections were considered.Analysis of beam column joint is done with ISLB 150,ISLB 75,ISMB 150, ISMB 175,Specimen CB1,CB 2,CB 3,CB 4 respectively. By finding the fire resistance time of different I sections can able to find the effective section and relationship between fire resistance and section steel property.

A. Details of specimen

The beam column joints had identical beam and column sizes.. The beams were 300 mm deep by 200 mm wide and columns were 300 mm deep by 300 mm wide. Ordinary Portland cement, The 28-day compressive strength of the concrete cube was 27.8 N/mm2. Steel bars of yield stress 421 N/mm2 and 445 N/mm2were used as main reinforcement and stirrup. The cover for the longitudinal bars was maintained at 30 mm for all the units. Adequate development lengths as per the code requirement were given for the beam longitudinal bars to take care of the pull out force. Column reinforcement consist of main bars of 20 mm diameter 4 numbers and ties of 8 mm diameter at 150 spacing and beam reinforcement consist of main bars 12 mm diameter 4 numbers ties of 8mm diameter a 150 spacing slab reinforcement consist of longitudinal bars of 8mm diameter at 140 spacing and distribution bars of 8 mm diameter at 250 mm spacing.The column height was 3800 mm and the total beam length was 3900 mm. Slab is of 2000 width 2000 length and 100 mm thickness. Section steels are different I sections. Details of the I sections are included in Table 1.The two tests were designed to investigate the effect of different I sections on the fire resistance of the steel reinforced beam column joint. The load level in the SRC beam,which is defined as m = PF/Pu; where PF is the longitudinal load applied on the beam and Pu is the bearing capacity of the beam at ambient temperature . k is the beam–column linear stiffness ratio, which is defined as k =[(EI)b/L]/[(EI)c/H]=.706, where (EI)beam and(EI)column are the flexural stiffness of beam and column respectively; L and H are

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the length of beam and the height of the column. The flexural stiffness EI=EsIs + EsbIsb +0 .6EcIc was determined according to Eurocode 4,9 where Es, Esb and Ec are the modulus of elasticity of the section steel, steel bars and concrete respectively, and Is, Isb and Ic are the moment of inertia for the section steel, steel bars and concrete cross-section respectively; n is the level of axial load in the column, which is defined as n =NF/Nu, where NF is the axial load applied in the column and Nu is the axial compressive capacity of the column at ambient temperature. In this test, Ncu and Pbu were taken as 4240 and 86kN, respectively. The SRC joints were designed based on the Chinese standard JGJ 138-2001.10

	DESIGNATION	SECTIONAL AREA	DEPTH	WIDTH OF FLANGE	THICKNESS OF FLANGE	THICKNESS OF WEB
CB 1	ISLB 150	9.01	150	50	4.6	3.0
CB 2	ISLB 175	10.28	175	50	4.8	3.2
CB 3	ISMB 150	19	150	80	7.6	4.8
CB 4	ISMB 175	24.62	175	90	8.6	5.5

B. Test procedure

1.The bottom end of the column was fixed, and the top end of the column was restrained against all directions except the vertical direction. The two ends of the beam were free from restraint. Axial load (NF) was applied to the column, and two vertical loads (PF) were applied to each end of the beam on each side of the joint step wise respectively. curve, and the fire was continued until the fire resistance 2. Heating phase: The applied loads on the column (NF) and beam (PF) were kept constant, and the space under the RC slab was heated, as marked in Fig. 2. and the fire was continued until the fire resistance .

C. Material properties

One kind of concrete was used to fabricate the composite joints. The mix proportions were: cement: 315 kg/m3; water: 175 kg/m3; sand: 744 kg/m3; coarse aggregate: 1071 kg/m3; fly ash: 75 kg/m3; and water reducer: 7.02 kg/m3. In the concrete mix, the fine aggregate used was silica-based sand; the coarse aggregate was carbonate stone. The average ambient temperature cube strengths at 28 days and at the time of test were 27.8 and 30.6 MPa, respectively. The modulus of elasticity (Ec) of the concrete at the time of test was 29200 MPa.

Steel. An elastic–plastic model was used to describe the constitutive behavior of the steel. The stress–strain model for steel at high temperatures given by Lie and Denham was adopted herein. Steel with the yield stress (fy) of 335 MPa at ambient temperature is selected as an example to show the stress against strain (s) response varied with temperatures. The initial modulus of elasticity could be determined by the slopes of the stress–strain curves at high temperatures. The Poisson's ratio for steel in the elastic stage is taken as 0.3 in the calculation. The residual stress was considered in the FEA modelling for welded I-section steel. The initial modulus of elasticity could be determined by the slopes of the stress–strain curves at high temperatures. The Poisson's ratio for steel in the elastic stage is taken as 0.3 in the calculation. The residual stress was considered in the FEA modelling for welded I-section steel.

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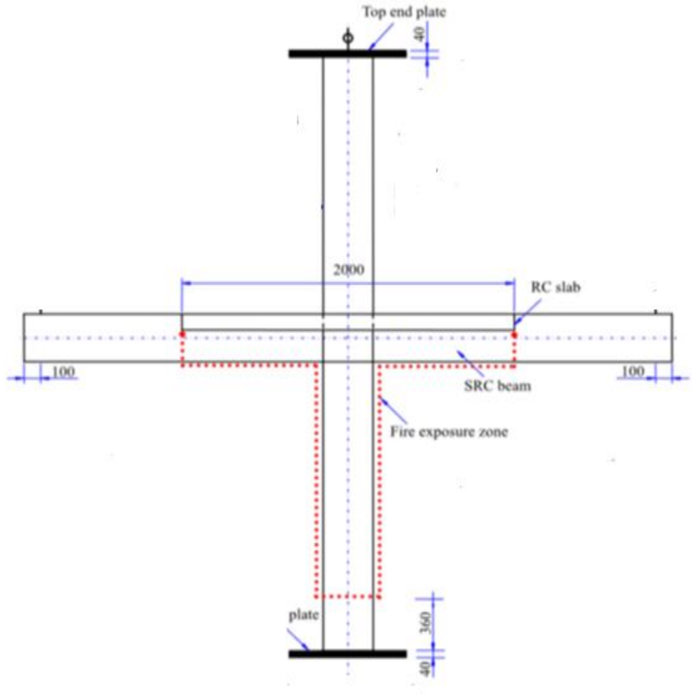

Fig.1.Beam column joint showing heating area

Concrete. The yielding surface and the description of the plastic behavior coming from the equivalent stress–strain relationships of concrete were defined. Typical stress–strain (c) curves of concrete with a cube compressive strength fcu =60 MPa at different temperatures using the model given by Lie and Denham are shown in Figure 7. The initial modulus of elasticity was also obtained by the slopes of stress–strain curves at high temperatures. Poisson's ratio for concrete changes at 1508C to reach 50% of its ambient value at 4008C and becomes zero at 12008C.

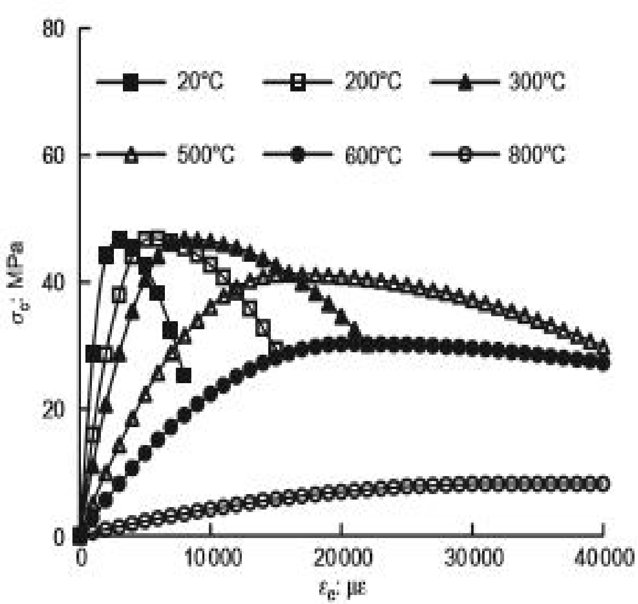

Fig.2.Typical stress- strain curves of steel

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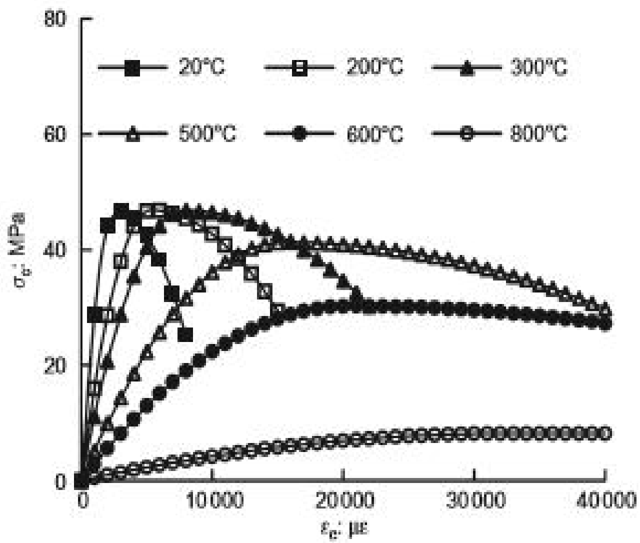

Fig.3.Typical stress-strain curves of concrete

D. Thermal properties of steel and concrete

The thermal properties for steel, including H-shaped steel and all the bars, were adopted according to the expressions provided by Lie and Denham (1993). Thermal properties of concrete are mainly affected by the mixture, type of aggregate and moisture content. Here, the carbonate aggregate concrete thermal model proposed by Lie and Denham (1993) were used in column, beam and slab. The effect of water on the thermal properties of concrete was taken into account using the method suggested by Lie (1994). This method assumes that all the water vaporizes at 100˚C, and the effect of water vaporization on the heat transfer is ignored, the influence of water on density and specific heat of concrete was considered by combining the properties of water into concrete, as shown below:

ρc 'cc'=.95 ρc cc+ .05 ρw cw T<100 degree Celsius

= ρc cc T>/= 100mdegree Celsius

where ρc ′ and cc ′ are the density and specific heat of concrete after modification; ρccc are the density and specific heat of concrete suggested by Lie and Denham (1993); ρw and cw are the density and specific heat of water and the value for ρw cw is taken as 4.2 × 106 J/(m3°C). In reality, the process of water vaporization absorbs a certain amount of heat and delays the temperature increase of the concrete for a while. Typically the time temperature curve of concrete members shows a small plateau at or slightly above 100°. Ignorance of this effect causes the the predicted temperature rises to be a few minutes faster than the measured temperatures.

The actions of heat convection and radiation were included as boundary conditions in the temperature field analysis as shown in Figure 5.5. The heat transfers from the environment to the surface of the composite joint during the heating phase, and then the heat transfers from the surface of the composite joint to environment when the specimen is hotter than the air during the cooling phase. For the inner of the composite joint, the heat transfers along the direction of temperature gradient. It is presumed that the initial temperatures of the air and the composite joint are 20°C before fire exposure. The simulation of boundary conditions in heating and cooling phases adopts the way suggested in Song et al. (2010), which has been used to calculate the temperature distributions of concrete filled steel tube (CFST) column to steel beam joints under fire including the cooling phase. The convective heat transfer coefficient and the surface radiation emissivity are taken according to ECCS (1988) to be 25W/(m2°C) and 0.5.

E. Result and discussion

Analysis of beam column joint with different I section was done. Directional deformations are found out which lead find the

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maximum downward deflection.. Directional deformations are found out which lead find the maximum downward deflection. Maximum downward deflections with respect to time is plotted in graph is shown as fig 4,5,6. Fire resistance for various specimens CB1 ,CB2, CB3, CB4 were found out as138min,152min,160min,172 min respectively.Max downward deflection of beam ends of various specimens were found out as 132 mm,112 mm,89mm 63 mm.Fire resistance time is the time for which a beam column joint can withstand the fire,which is increasing as we move from ISLB 150 to ISMB 175. Maximum downward deflections are occuring on right beam ends due to step wise laoding.From the results obtained ,most effective section is CB4 which is the beam column joint with ISMB 175. The cross sectional area of the beam column joint ISMB 175 is greater than the other sections used.

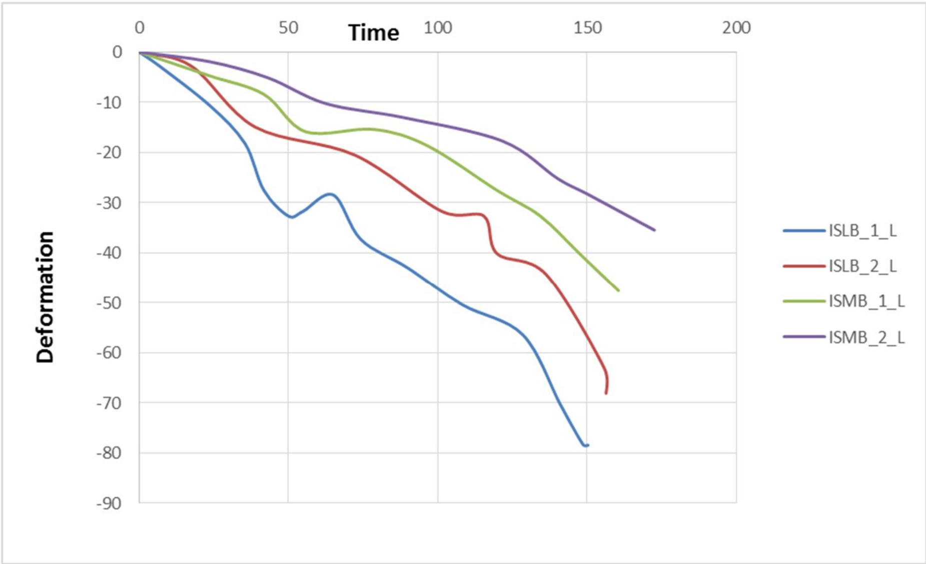

Fig.4.Time versus deformation graph of left beam

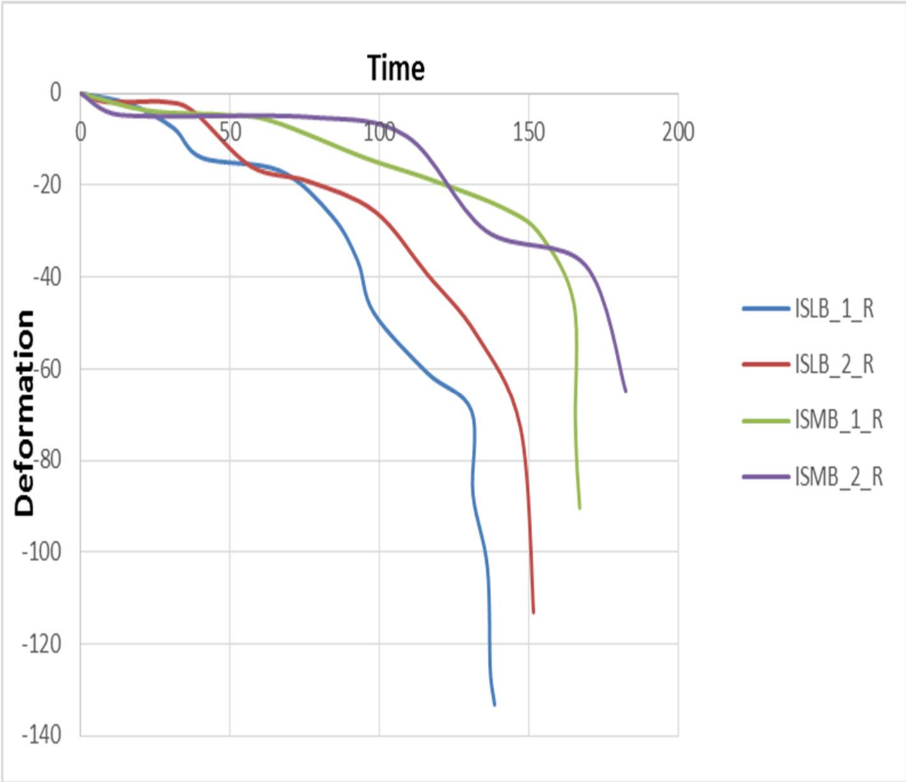

Fig.5.Time versus deformation graph of right beam

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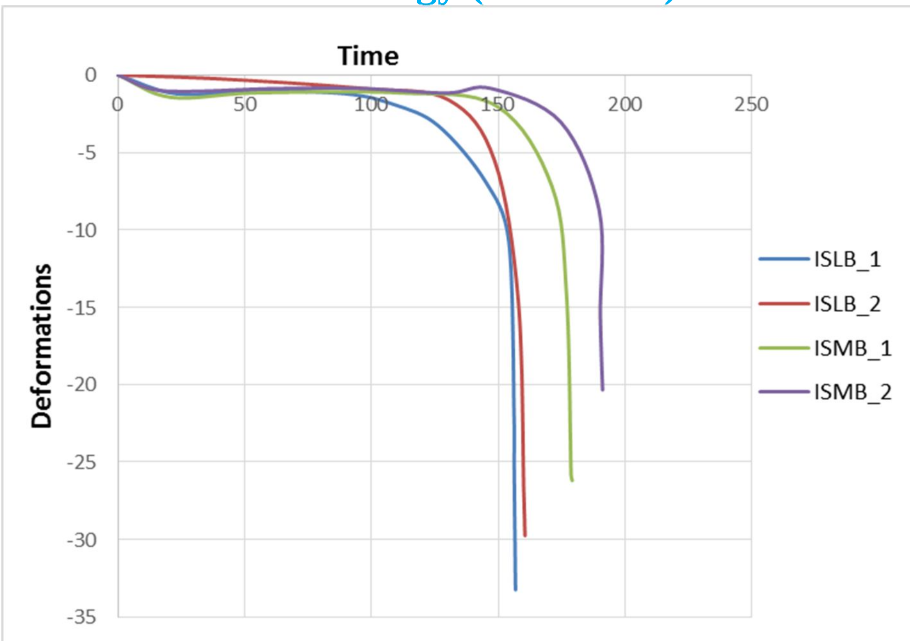

Fig.6.Time versus deformation graph of column

III. POST FIRE BEHAVIOUR OF SRC BEAM COLUMN JOINT

A. Test procedure

For Specimens CB 4, after the fire resistance time were reached, the specimens were cooled down to room temperature. In the cooling phase, the loads applied on the column and beam were kept stable until the specimen cooled to an ambient temperature.

Post fire phase: After the specimen cooled to the ambient temperature, the loads on column both the left and right beam segments were applied at the same time until the joint failed at an ultimate load (Ppu) on each beam segment.

B. Result and discussion

Column or beam deformation suddenly increased in cooling phase .Max deformation was 103 mm, which is nearly double the deflection of same beam column joint in the heating phase. Deformation with respect time graph of CB 4 is shown in fig 6.2

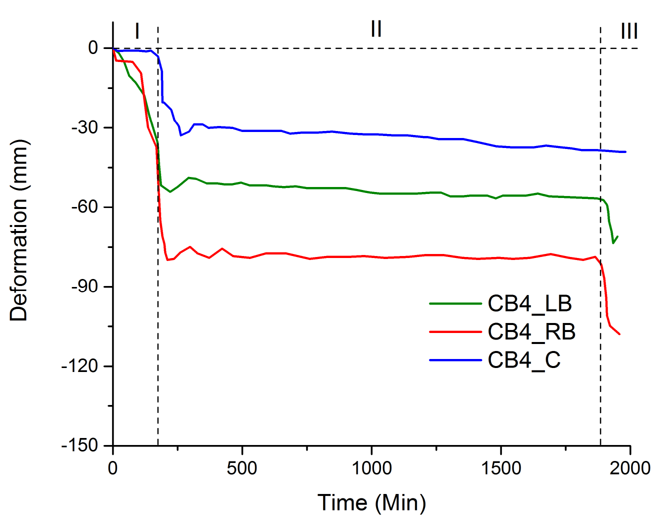

Fig.7.Time versus deformation graph pf CB 4 on post fire test

IV. CONCLUSIONS

Tests were carried out on SRC beam– column joints subjected to the fire with different I section steels as structural steel reinforcement in this paper. A finite-element analysis modelling was presented for the analysis of the composite joints under fire. A comparison of results calculated using this model shows good fire resistance for beam column joint as area of I section increases. The mechanism of the SRC joint under fire was analysed using the FEA modelling.

The post fire experimental results clearly indicate that the increment of column or beam deformation occurring in the cooling phase is much larger than that occurring in the heating phase, which demonstrates the potential failure of SRC joints in the cooling phase. In addition, the increments of column axial deformation changed from 5 to 10 mm as the heating times increased from 45 to 90min ,but the influence of heating times on beam deformations in the cooling phase was not obvious.

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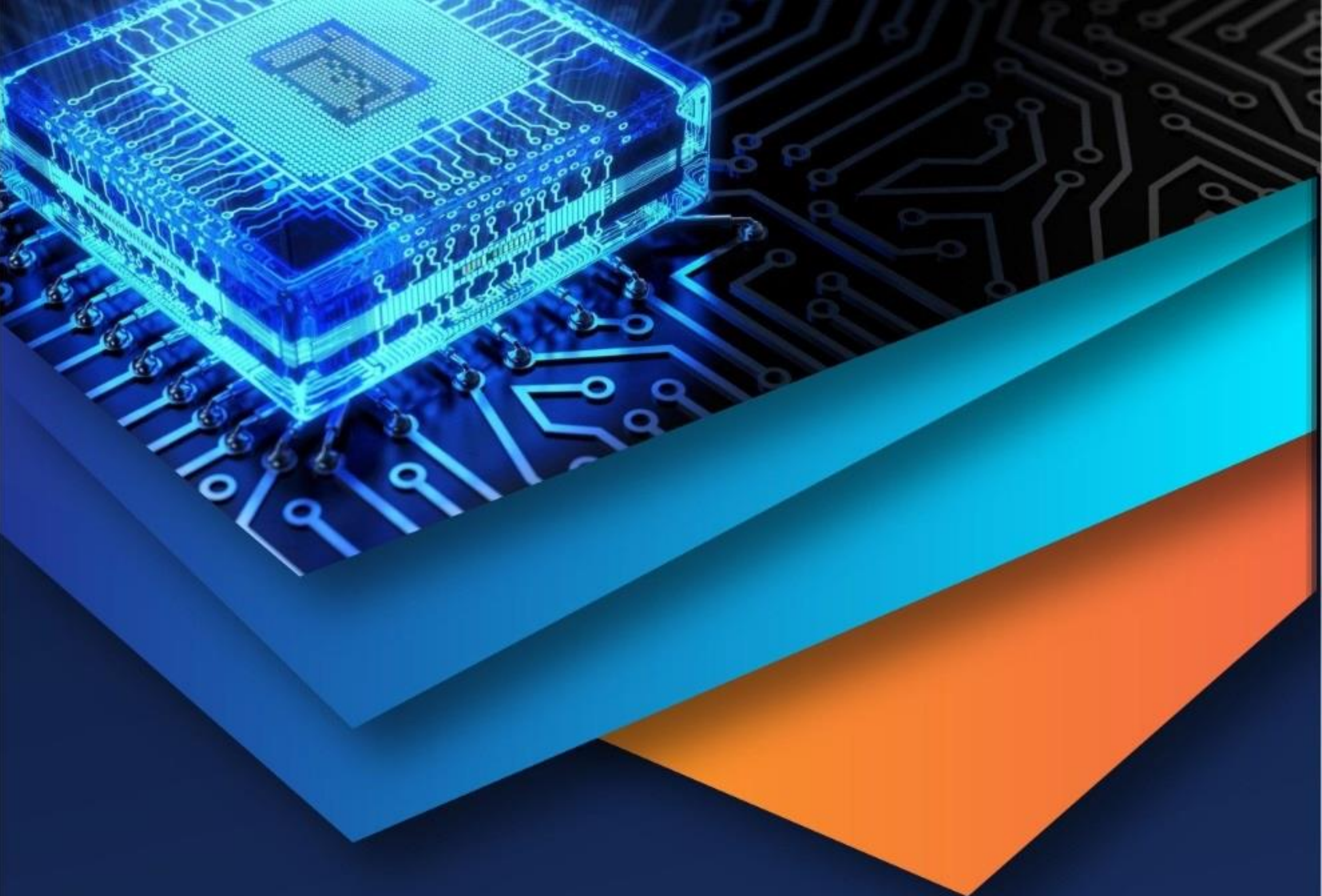

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