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Structural Behaviour of Welded I-Beam to Box-Column Connections under Dynamic Loading

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Abstract: Hollow square steel sections have superior performance than the conventional steel sections in the construction of moment resisting frames. They have a greater capacity to resist biaxial bending and also provide a higher torsional stiffness and resistance than conventional steel sections. But the use of such sections are limited as less research works have been carried out on I Beam to Box Column connections. The objective of the present study is to develop a suitable and economic connection detail between I beam and Box column. Non-linear static analysis has been performed for three configurations of beam column connections. The stress concentration, strength and ductility for each connection has been studied and the best connection is recommended for I beam to Box column joint. The connections were checked for seismic provision requirements for special moment resisting frames. In order to understand the behaviour of a steel frame under seismic conditions, non-linear static pushover analysis has been performed for steel frames using the obtained suitable connection. The natural frequencies and mode shapes were calculated for the steel frames using modal analysis and the performance of these steel frames under seismic ground acceleration of Northridge earthquake has been examined.

Keywords: I beam to box column connection, Non-linear static analysis, Pushover analysis, Modal analysis, Time history analysis

I. INTRODUCTION

Steel buildings are widely accepted all over the world due to their easy installation and cost efficiency. Conventional steel sections such as I sections are commonly used in the design of beam column connections in steel structures. But hollow square steel sections have superior performance than the conventional steel sections in the design of moment resisting frames. They provide good aesthetic appearance and higher strength to weight ratio. Box columns are designed in high risk seismic areas due to its excellent strength in terms of equal loading in all directions. Their large bending capacity about any axis makes these sections more efficient than wide flange sections in flexural and compression members such as beam-columns. Since the cross section of such columns are closed, they provide a higher torsional stiffness and resistance than the conventional steel sections.

Although these hollow square columns have the above mentioned advantages, finding a proper I Beam to Box Column connection is still under investigation. Several studies have been carried out and new connection details have been proposed for of I-beam to wide flange connections since the 1994 Northridge earthquake. Keshavarzi et al. [1] proposed a new moment connection for I beams to box columns which consists of a vertical plate passing through the column and welded to the column flanges, and beams connected to the plate named the Through Plate. Saeed Erfani et al. [2] proposed a new connection configurations in which a short stub beam is used to connect I beams to box columns by bolted end plate connection. Rupen Goswami and Murty [3] conducted a study on an externally reinforced I beam to box column seismic connection in which an inclined rib-plated collar-plated configuration with web plates is used to ensure planar continuity between I beam and box column webs. Gholami et al. [4] conducted analytical and experimental studies on the cyclic behaviour of flange plate connection between a steel beam and a welded box column.

The objective of the present study is to develop a suitable and economic connection detail between I beam and Box column. Non-linear static analysis has been conducted for three different connections to evaluate its behaviour. The stress concentration, strength and ductility for each connection has been studied and the best connection detail is recommended for I beam to Box column joint. The moment rotation characteristics for these connections were developed. In order to study the behaviour of a steel frame under seismic conditions, non-linear static pushover analysis has been performed for steel frames developed using the obtained suitable connection. The natural frequencies and mode shapes corresponding to the first two natural mode of vibration is calculated for the steel frames using modal analysis and the performance of these steel frames under seismic ground acceleration of Northridge earthquake has been checked.

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II. CAPACITY DESIGN CONCEPT

Beam to column connections in steel moment resisting frames MRFs are designed based on the capacity design concept. This is to ensure that these connections are able to sustain large lateral drift levels under seismic shaking through elastic actions only, pushing inelastic actions into the beam. Thus, connection elements and welds are designed corresponding to beam over-strength capacity.

The basic concept of capacity based design of structures is the spreading of inelastic deformation demands in a structure in such a way so that the formation of plastic hinges takes place at predetermined positions and sequences. In multi storey multi bay reinforced concrete frames, plastic hinges are allowed to form only at the ends of the beams. To achieve this flexural capacity of column sections at each joint are made higher than the joining beam sections [5]. This will eliminate the possible sway mechanism of the frame. The capacity design is also the art of avoiding failure of structure in brittle mode. This can be achieved by designing the brittle modes of failure to have higher strength than ductile modes. Shear failure is brittle mode of failure hence shear capacity of all components are made higher than their flexural capacities.

III. APPLICATION OF NON-LINEAR STATIC ANALYSIS

The performance of a structure can be determined using non-linear static analysis. The purpose of non- linear static analysis is to determine the performance of the structure by determining the strength and deformation capacity. In the present study, the analysis is performed by applying monotonically increasing loads until the failure occurs. As the load is increased, weak links or failure modes will form at the beam column connection. At each step, a graph is plotted between the total shear force at the fixed end and the displacement at the free end of the beam. The output generates a curve which plots a strength based parameter against deflection. Fig. 1 represents the typical force vs deformation curve.

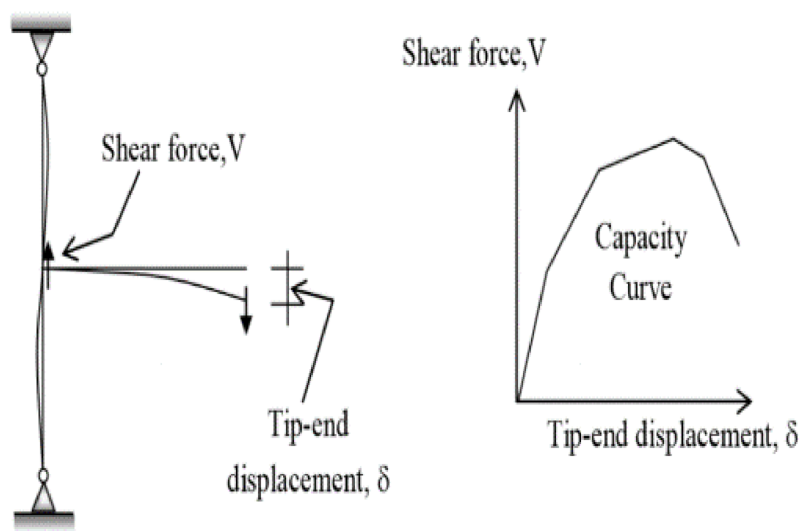

Fig. 1 Force vs deflection curve

IV. MOMENT ROTATION CHARACTERISTICS OF I BEAM TO BOX COLUMN CONNECTIONS

The Moment resistance developed at the column face is plotted against the inter storey drift angle to study the seismic behaviour of the selected connections. From the hysteresis graph, we can determine the maximum drift angle the connection can withstand. According to IS 800:2007, the measured flexural resistance of the connections determined at the column face, should be equivalent to atleast 0.80Mp of the connected beam at inter story drift angle of 0.04 rad where Mp is the plastic moment capacity of the connected beam. The hysteresis curve is plotted for each connection and their results are compared. Similarly for determining the seismic behaviour of different connections, cyclic loading was applied at the beam tip. This was applied in accordance with SAC loading protocol as shown in Fig. 2 [4]. This loading protocol was developed to obtain the behaviour of beam column moment connections in moment resisting frames.

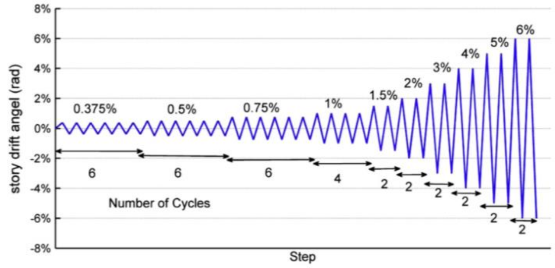

Fig. 2 SAC Loading Protocol

V. SEISMIC ANALYSIS OF I BEAM TO BOX COLUMN CONNECTIONS

Buildings are designed and detailed to develop favourable failure mechanisms that possess specified lateral strength, reasonable stiffness and, above all, good post-yield deformability. Ductility of a building is its capacity to accommodate large lateral deformations along the height. It is quantified as the ratio of maximum deformation to the yield deformation. Thus, a ductile building exhibits large inelastic deformation capacity without significant loss of strength capacity [14]. The capacity of a structure to withstand seismic actions can be determined by performing non-linear static pushover analysis. The non-linear static pushover analysis is a popular method for seismic performance evaluation of existing and new structures. This analysis will provide adequate information on seismic demands imposed by the design ground motion on the structural system and its components. A plot of the total base shear versus top displacement in a structure is obtained from this analysis that would indicate any premature failure or weakness of the structure. Another method to determine the seismic behaviour of a structure is modal analysis. The frequencies at which vibration naturally occurs and the mode shapes which the vibrating system assumes are the properties of the system, and can be determined using modal analysis. The deformed shape of the building associated with oscillation at fundamental natural period is termed as its first mode shape. Similarly, the deformed shapes associated with oscillations at second, third and other higher natural periods are called second mode shape, third mode shape and so on respectively.

VI. FINITE ELEMENT ANALYSIS OF I BEAM TO BOX COLUMN CONNECTIONS

Inorder to study the performance of different beam to column connections, three-dimensional finite element models were developed in ANSYS. The beam, column and connections were modelled using SOLID 185 element. This element is defined by eight nodes having 3 degrees of freedom at each node ie, translations in the nodal x, y and z directions. The element has plasticity, stress stiffening, large deflection and large strain capabilities.

The connection consists of a wide flange beam- ISWB-350x200x8x11.4 connected to a hollow square steel column of size 400x400x20x20. Von- mises yield criteria was employed to consider the material non linearity. The bilinear stress strain curve was adopted for modelling the materials as shown in Fig. 3. Table I represents the material properties for both beam and column.

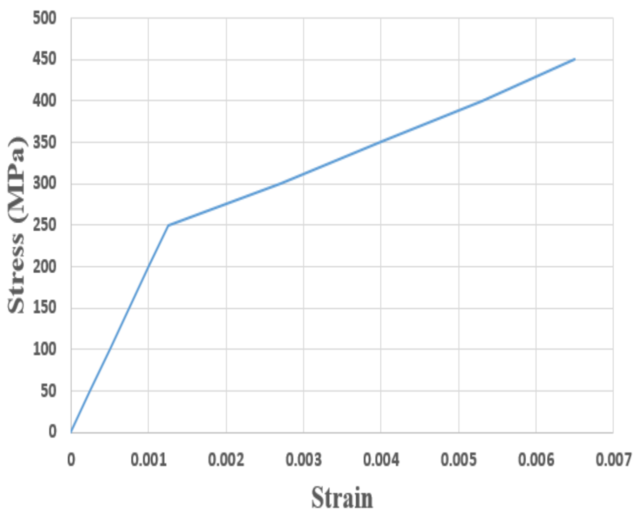

Fig. 3 Bilinear stress-strain relationship of beam column model

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Table I Material Properties

Properties	Quantity
Density	7850 kg/m^3
Young's Modulus	200 GPa
Poisson's ratio	0.3
Yield Stress	250 MPa
Failure Stress	460 MPa

The study aims in developing a suitable and economic connection detail between I beam and Box column. Three different connection configurations were tested to evaluate its behaviour. The top and bottom end of column was provided with hinged support. Monotonically increasing load is applied at the beam tip. The stress concentration, strength and ductility for each connection is studied and the most suitable connection detail is recommended for I beam to Box column joint.

A. Simple Welded Connection

In the simple welded connection, the beam is directly welded to the column flange. Here the beam web and flanges are connected to the column. The FE model of the connection is shown in Fig. 4.

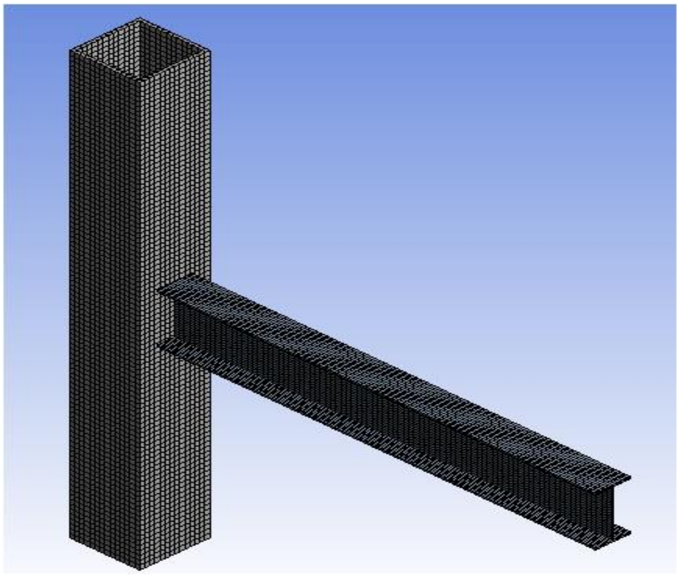
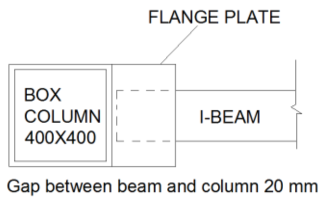


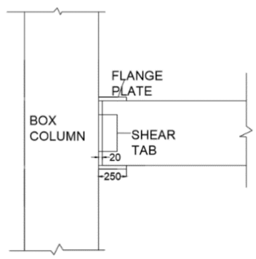
Fig. 4 FE model of Simple welded connection

B. Flange plate connection

In this connection, beam is connected to the column by using top and bottom flange plates. Shear tabs are also provided to connect the beam web to the column. The thickness of the flange plate is 20mm. The flange plate is provided to the full width of column in order to transfer the forces from the beam towards the column webs. Study has been conducted by varying the length of the flange plate to obtain the proper length. Flange plates of length 200, 250 and 300 mm are considered. The plan and elevation view of the connection is shown in Fig. 5 and the finite element model of the connection is shown in Fig. 6.



a) Plan



b) Elevation

Fig. 5 Plan and elevation of the flange plate connection

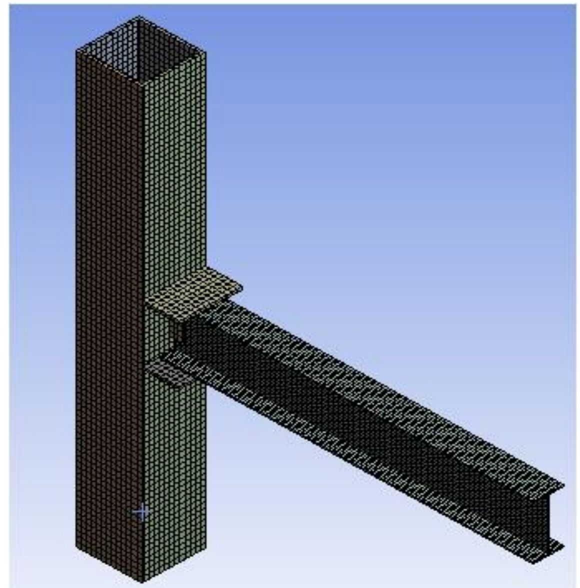


Fig. 6 FE model of flange plate connection

C. T Stiffener Connection

External stiffeners are provided in the form of T stiffeners on the beam flanges connecting the column. This connection will transfer the forces from beam towards the column flanges and web very effectively. The width of beam flange at the connection compared to the column width is an important factor affecting the moment rotation behaviour of the connection. By providing the T stiffeners, the beam width will be same as that of the column width. The finite element model of the connection is shown in Fig. 7.

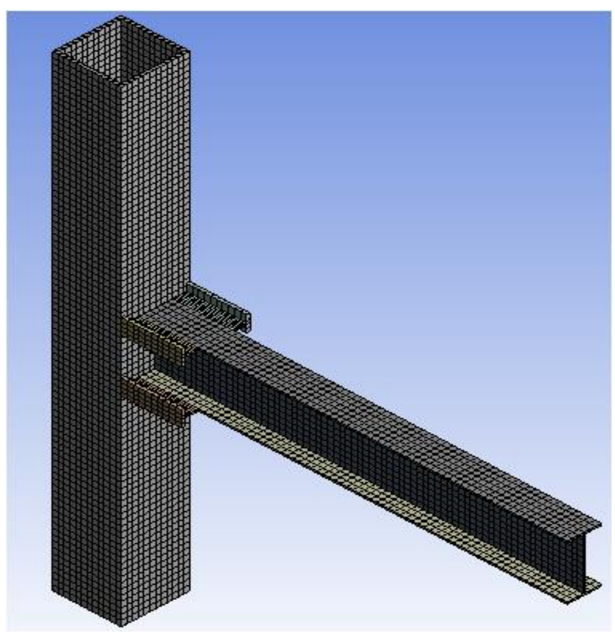


Fig. 7 FE model of T stiffener connection

VII. RESULTS AND DISCUSSION

A. Non-linear Static Analysis

1) Simple welded connection: The model was analysed by applying load incrementally at the beam tip to plot the shear force vs deflection graph. By performing the non-linear static analysis, it is found that major stress was concentrated at the beam-column joint of the structure as shown in Fig. 8 indicating that plastic hinge is formed at the column which is not good for a seismic connection. For a good seismic connection, the formation of plastic hinge should be away from the beam column joint. Maximum deformation at the time of failure is 161.19 mm. The maximum strength for this connection is found to be 48kN. Ductility factor for the basic connection is 3.03. The Force deflection curve for this connection is plotted as shown in Fig. 9.

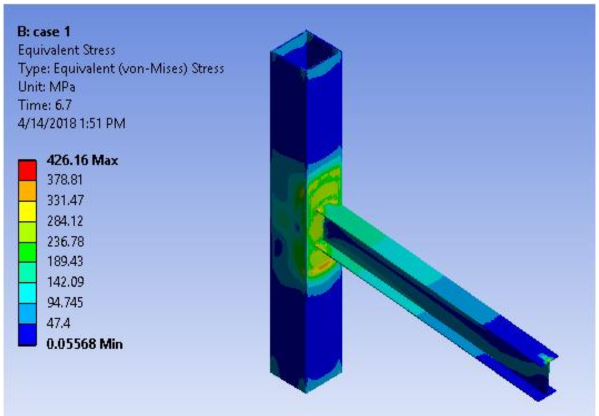


Fig. 8 Stress distribution at failure for simple welded connection

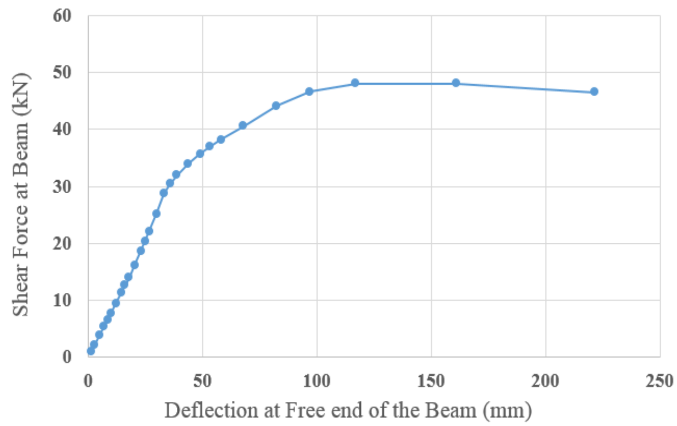


Fig. 9 Force vs deflection curve, Simple welded connection

2) Flange Plate connection: In this connection, the plastic hinge is formed at the nose of the flange plate which is away from the beam-column connection with high stress concentration indicating the failure of connection at the beam section. Since the stress concentration at the column face is minimum, the flange plate connection can be used as a good seismic connection. Fig. 10 shows the stress distribution in this connection.

The force vs deflection curves are plotted for the connection with different flange plate length of 200mm, 250mm and 300mm. The results obtained is shown in Fig. 11. From the obtained graph, it is clear that the connection is stronger when a flange plate of 250 mm is used. A longer flange plate will increase the beam plastic rotation demand and can lead to a greater chance of failure at the plastic hinge location. Thus a flange plate of 250 mm was used for the study. The maximum strength is obtained as 60 kN and also the connection achieves a high ductility factor of 4.80.

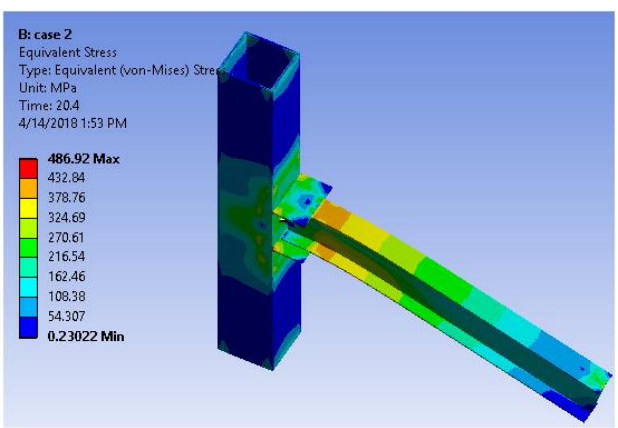


Fig. 10 Stress distribution at failure for flange plate connection

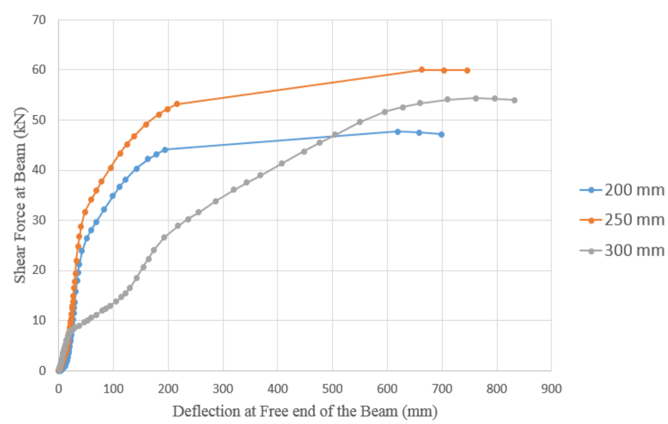


Fig 11. Force vs deflection curve for varying length of flange plate

3) T Stiffener Connection: The beam column connection with T stiffeners behaves better when subjected to non-linear static pushover analysis. In this connection, the stress developed at the column is minimum and a maximum stress of 360.54 MPa is found to occur at the beam. From the distribution of stresses, it is observed that maximum stress concentration was found at the beam flange section connecting the stiffeners which is away from the beam column connection. Thus such a connection satisfies the

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criteria of capacity design concept. Here, since the plastic hinge is located at the beam, the damage during a seismic action will first occur at the beam by making the column safer. The stress distribution of connection is shown in Fig. 12.

Beam column connection with T stiffener performs the best in terms of shear strength and ductility among the selected connection details. Maximum shear strength obtained in this configuration is 65.14 kN. This connection also ensures a very good ductility behaviour with a ductility factor of 4.94. The ultimate shear force is increased by 35% with respect to the simple welded connection. The shear force vs deflection curve for the T stiffener connection is shown in Fig. 13.

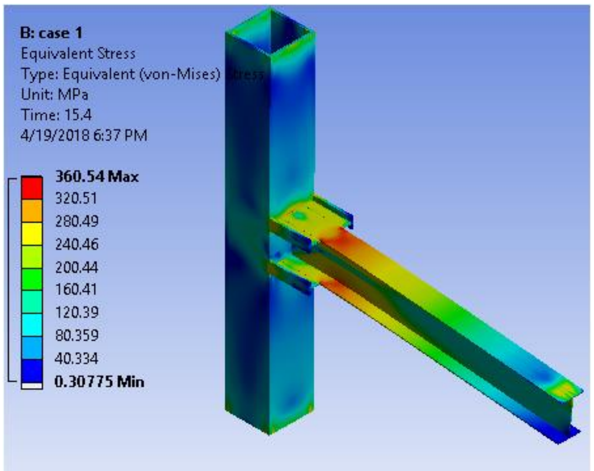

Fig. 12 Stress distribution at failure for T stiffener connection

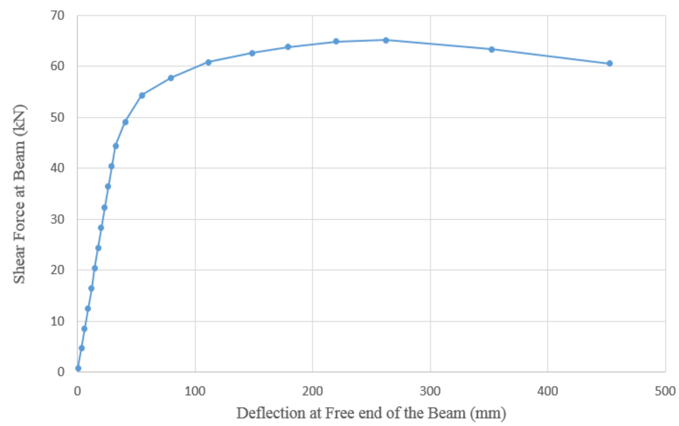

Fig. 13 Force vs deflection curve of T stiffener connection

B. Moment Rotation Characteristics of the Connections

The moment resistance developed at the column face is plotted against the inter story drift angle to study the seismic behaviour of the selected connections. From the hysteresis graph, we can determine the maximum drift angle the connection can withstand. According to IS 800:2007, the measured flexural resistance of the connections determined at the column face, should be equivalent to atleast 0.80Mp of the connected beam at inter story drift angle of 0.04 rad where Mp is the plastic moment of the connected beam. The hysteresis curve is plotted for each connection and their results are compared. Inorder to check the seismic provision requirements of the connections, the plastic moment capacity of the connected beam is calculated as follows:

$$M_p = \sigma_y \text{ x } Z_p \qquad (7.1)$$

For ISWB-350, Z_p = 995490 mm^3 , σ_y = 250 N/mm^2

M_p for the section is 246 kNm and 0.8 M_p = 196.8 kNm

The moment corresponding to 4% drift angle is determined to check whether it attains atleast 80% of the plastic moment capacity of the beam. Results are drawn from the plotted hysteresis curves. It can be concluded that both the flange plate connection and T stiffener connection satisfies the seismic code provision for special moment frame structures. The results obtained are shown in Fig. 14 – Fig. 16.

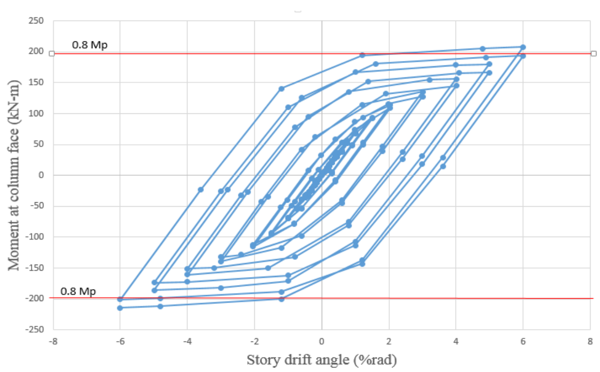

Fig. 14 Hysteresis curve for simple welded connection

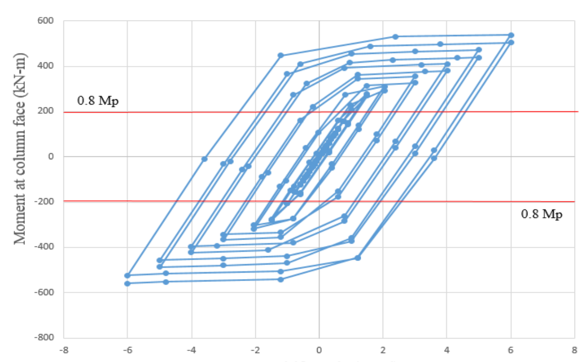

Fig. 15 Hysteresis curve for flange plate connection

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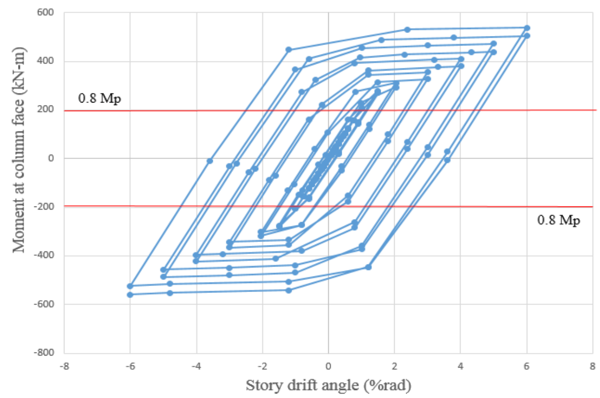

Fig. 16 Hysteresis curve for T stiffener connection

From the results, it is observed that the beam column connection with T stiffener performs the best under seismic conditions. The flange plate connection and the T stiffener connections can be used as a part of special moment resisting frame structures. In simple welded connection, the moment at the column face at 4% rad story drift angle is less than 80% of the plastic moment capacity of the connecting beam. Therefore this connections does not satisfy the seismic provision requirements for special moment resisting frames. The moment rotation characteristics of the I beam to box column connections are shown in Table II.

Table II Moment Rotation Characteristics of the Connections

Connection detail	Moment at 4% rad story drift (kNm)	Moment at full cycle (kNm)
Simple welded connection	145.23	214.1
Flange plate connection	373.42	567.81
T stiffener connection	422.9	560.87

C. Non-linear Static Pushover Analysis of Steel Frames

From the results of non-linear static analysis and the moment rotation behaviour of the connections, it is observed that the T stiffener connection performs best in terms of higher strength and ductility. This connection also satisfies the seismic provision requirements for the special moment resisting frames according to IS 800:2007. Therefore non-linear static pushover analysis has been conducted on a two storey multi bay steel frame with T stiffeners at the beam column joints. Another two storey muti bay steel frame with simple welded connection at the beam column joints was also modelled and is used for comparison. The results of non-linear static pushover analysis on these two steel frames were compared. The geometry of the two steel frames are shown in Fig. 17 and Fig. 18.

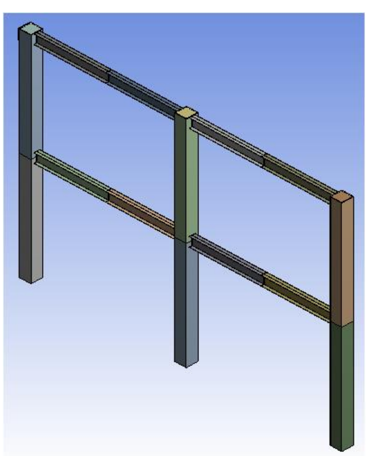

Fig. 17 Two storey steel frame with simple welded connection

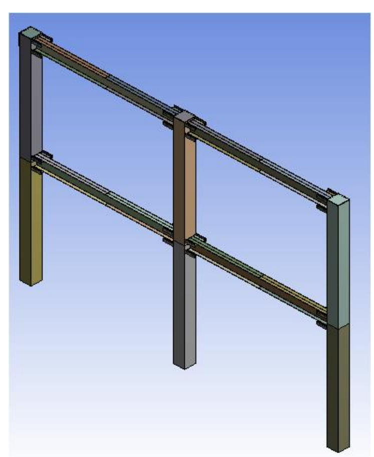

Fig. 18 Two storey steel frame with T stiffener connection

The steel frame with T stiffeners provided at the beam column connection performs best in terms of strength and ductility. Maximum strength of 760 kN is obtained in the steel frame with simple welded connection, while the steel frame with T stiffener connections achieves a high strength of 980 kN with a very good ductile behaviour. The pushover curves for the steel frames with simple welded connection and T stiffener connection are shown in Fig. 19 and Fig. 20 respectively. Table III represents the results obtained from the pushover analysis for both the steel frames.

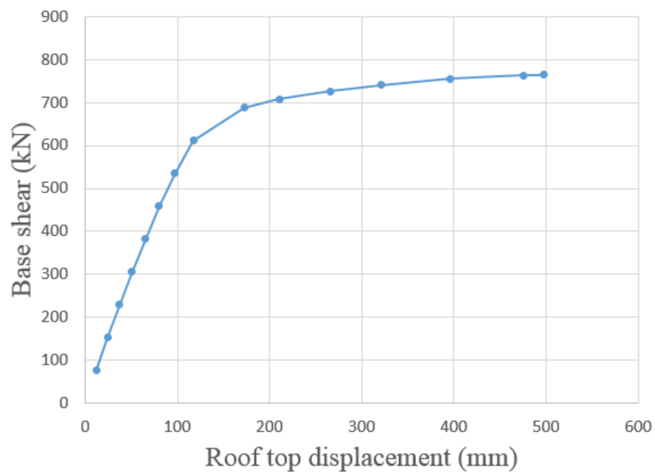

Fig. 19 Pushover curve for steel frame with simple welded connection

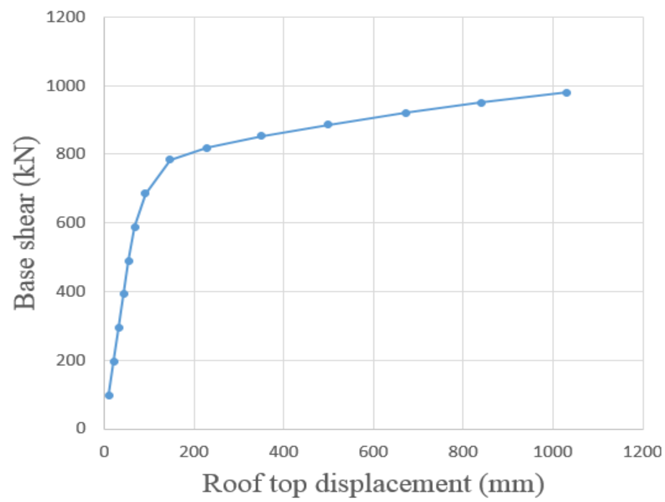

Fig. 20 Pushover curve for steel frame with T stiffener connection

Table III Performance of Steel Frames by Pushover Analysis

Steel frame type	Ultimate strength (kN)	Yield strength (kN)	Ultimate deformation (mm)	Yield deformation (mm)
Simple welded connection	760	610	496	150
T stiffener connection	980	720	1030	138

D. Modal Analysis of Steel Frames

In the present study, modal analysis is performed for the two steel frames constructed using simple welded connection and with T stiffener connections. From the analysis, the improved performance of the structure due to the application of T stiffeners in the beam column connection can be obtained. The mode shapes and frequencies corresponding to the first two modes of vibration is determined from the analysis. The steel frame with T stiffener connection have a higher frequency than the simple welded steel frame. The mode shapes obtained are shown in Fig. 21 and Fig. 22. Similarly the corresponding frequencies obtained are shown in Table IV.

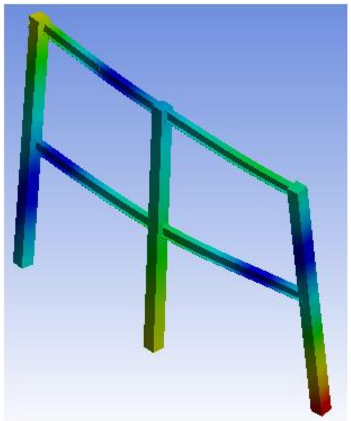

Fig. 21 Mode shape of Steel frame with simple welded connection

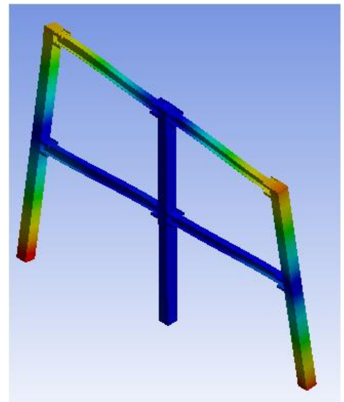

Fig. 22 Mode shape of Steel frame with T stiffener connection

Table IV Natural Frequency of Steel Frames

Steel frame	Frequency (Hz), Mode 1	Frequency (Hz), Mode 2
Simple welded connection	1.618	3.634
T stiffener connection	2.168	4.249

E. Non-Linear Time-History Analysis of Steel Frame

Non-linear time history analysis is performed for the two steel frames. The behaviour of the structures subjected to Northridge earthquake motion is studied. The Northridge earthquake was occurred on 17th January 1994 at the city of Los Angeles in California. It had a moment magnitude of 6.7 which produced ground acceleration that was the highest ever instrumentally recorded in an urban area in North America. The peak ground acceleration PGA of the Northridge earthquake is 8.268 m/s^2 at the time of 4.22 sec.

The ground acceleration vs time data of the Northridge earthquake is used as the input parameter. The excitation up to first five seconds is used for the study and is shown in Fig. 23.

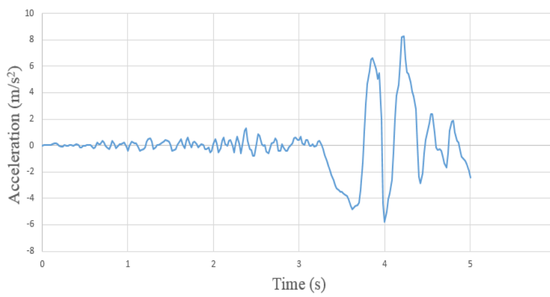

Fig. 23 Ground acceleration Vs time, Northridge earthquake, 1994

After performing the time history analysis, the response of the two steel frames were determined. The maximum deformation of the frames and the distribution of stresses during the earthquake is obtained. The deformation and stress distribution of the steel frame with T stiffener connection is compared with the results obtained in the steel frame with simple welded connection. Fig. 24 and Fig. 25 indicates the deformation and stress distribution of the steel frame with simple welded connection. Fig. 26 and Fig. 27 indicates the deformation and stress distribution of the steel frame with T stiffener connection.

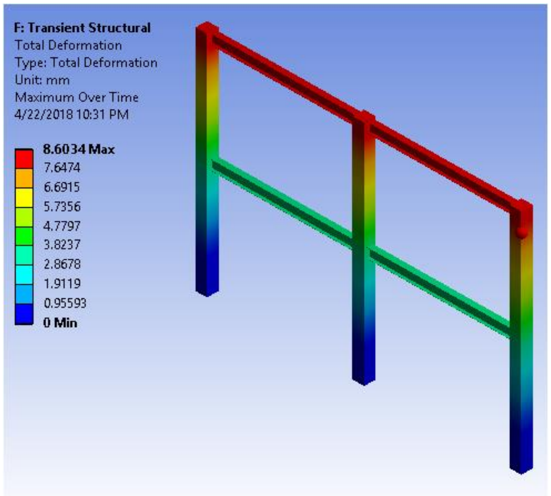

Fig. 24 Deformation contour of the steel frame with simple welded connection

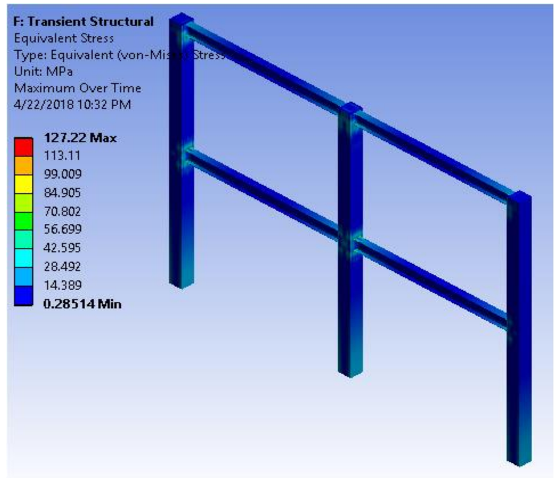

Fig. 25 Stress distribution of the steel frame with simple welded connection

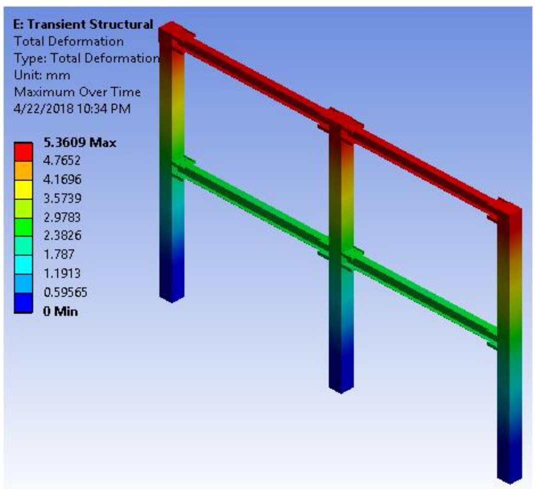

Fig. 26 Deformation contour of the steel frame with T stiffener connection

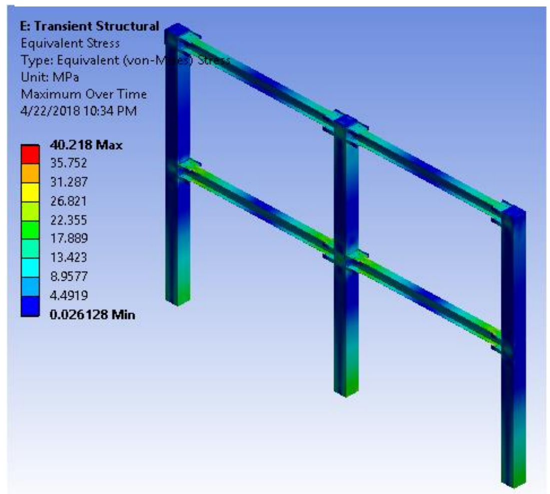

Fig. 27 Stress distribution of the steel frame with T stiffener connection

It is observed that the application of T stiffeners in the beam column connection helps in reducing the deformation at the top storey level. The maximum deformation obtained is 8.60 mm in frame with simple welded connection. While the steel frame with T stiffener connection has a lesser deformation of 5.36 mm. Similarly, the use of stiffeners also helped in reducing the stress concentration in the beam column connection. In simple welded connection, higher stress of 127.22 MPa was found to occur. Some amount of stress was distributed at the column face also. While in T stiffener connection, the maximum stress was reduced to 40.22 MPa. Moreover the maximum stress is concentrated at the end of T stiffeners which is at some distance apart from the column face. The results obtained from the time history analysis is presented in Table V.

Table V Performance of Steel Frames Subjected to Time-History Analysis

Steel frame	Maximum deformation (mm)	Maximum equivalent stress (MPa)
Simple welded connection	8.60	127.22
T stiffener connection	5.36	40.22

VIII. CONCLUSIONS

Non-linear static analysis has been performed for different I beam to box column configurations and the following conclusions were made:

A. Load carrying capacity and ductility is highly sensitive to the type of connection used.
B. T stiffener connection performs best in terms of shear strength and ductility
C. Ultimate shear strength of the joint found to vary from 48 kN in simple welded connection to 65 kN in T stiffener connection with an increase of 35%
D. T stiffener connection achieves a good ductility factor 4.94.
E. The plastic hinge formation is found away from the column face in both flange plate and T stiffener connection.

The moment rotation characteristics of the three connections were examined under SAC cyclic loading protocol. Both flange plate connection and T stiffener connection satisfies the requirements specified in the code IS 800:2007.

In order to evaluate the behaviour of beam column connections under seismic conditions, steel frames were modelled using simple welded connection and T stiffener connection. These frames were then subjected to non-linear static pushover analysis, modal analysis and time history analysis and the following results were obtained:

1) By performing non-linear static pushover analysis, the steel frame with T stiffener connection attains a higher base shear capacity of 980 kN than the simple welded steel frame with an increase of 23%. The steel frame with T stiffener connection also exhibits a good ductile behaviour.
2) The natural frequencies and mode shapes corresponding to the first two modes of natural vibration were obtained and the results are compared. Higher frequency was obtained in the steel frame with T stiffener connection.
3) The application of T stiffener in the beam column connection helps in reducing the deformation and stress concentration of the frame when subjected to time history analysis. The deformation is reduced by 38% when the T stiffeners are provided at the beam column connection. Similarly, the stress concentration at the connection was found to reduce by 3 times than the steel frame with simple welded connection.

From the results, it can be concluded that the application of T stiffeners helps in improving the overall dynamic performance of the structure by effectively transferring the forces equally towards the column flanges and webs from the beam and thereby increasing the overall strength of the structure.

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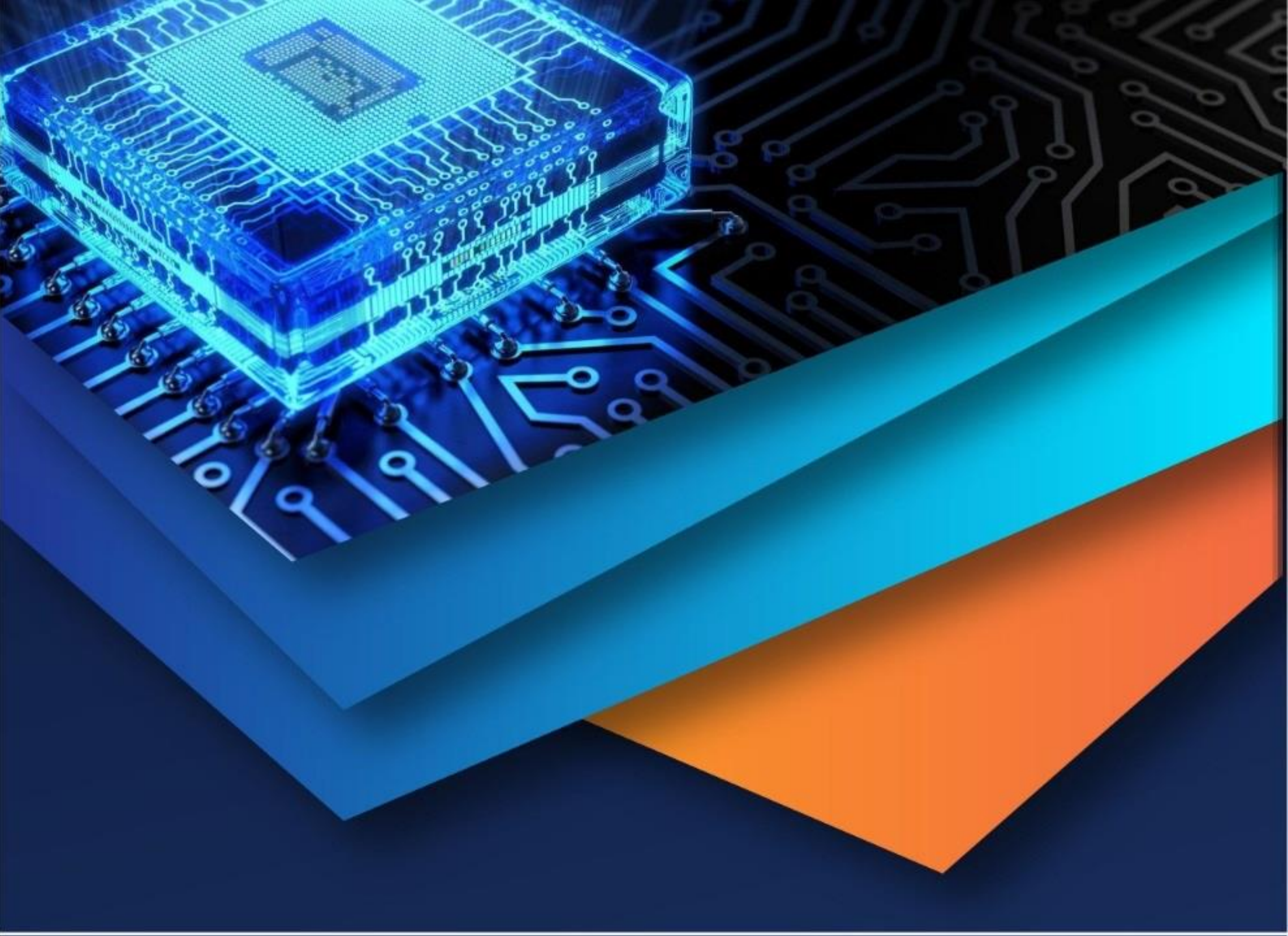

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