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Comparative Study on behaviour of Anti-Collapse Steel Beam Column Joint under Dynamic Loading with Static Loading: A Review

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ABSTRACT: In structural engineering, the ability of a structure to be maintained and serviced is crucial. Any structural component may fail as a result of unintentional activity, which could lead to a progressive collapse. Various methods are employed to reduce gradual collapse. One of such techniques is to study a connection that prevents collapse. Analysing the designing, however, is also crucial. A fundamentally different technique is required to analyse the beam-column connection against progressive collapse under the column removal scenario. In this paper, we discussed in general terms about that strategy.

KEYWORDS: Catenary Action, Progressive Collapse, Dynamic Increasing Factor, Flexural Action, Non Linear Static,

I. INTRODUCTION

Each component of structure subjected to continuous loading throughout its life. Due to any reason if any component gets damaged it will majorly affect the serviceability of structure. The major effect is progressive collapse which means due to failure of one single component it leads to complete collapse of structure because of continuous loading acting on structure. To avoid this anti-progressive collapse structures are designed now a days. For any structures beam and columns are main components of load transfer. Due to any reason if column fails then the load carried by column is not transferred properly to ground and this un-transferred load adversely affects the other components of structure eventually leads to complete failure. To overcome this, un-transferred load must be evenly distributed to adjacent load carrying components up to permissible limit and in proper manner. For this purpose the beam column connection must be designed in such a manner that after failure of column it will not collapse suddenly and transfer the coming load effectively on adjacent columns.

There are many types of joints invented which are anti-progressive collapse and performs satisfactory in static loading. But it is not possible that every time structure will experience static loading. Sometime dynamic loading occurs on structure so an anti-collapse joint which will perform satisfactory under dynamic loading is future demand

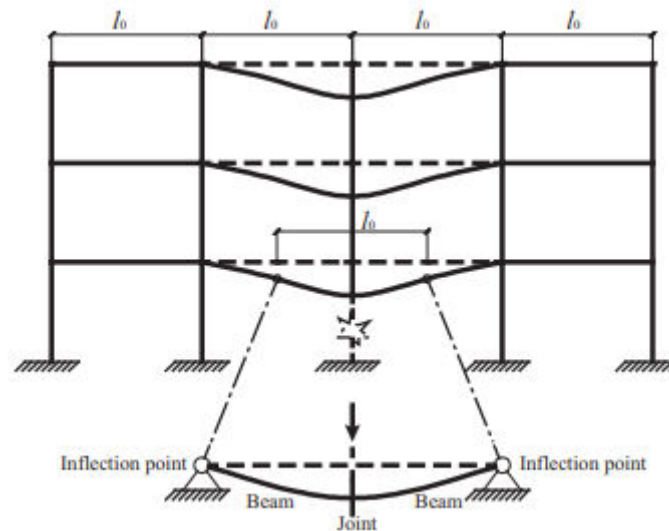


Figure 1: Progressive Collapse Condition

II. LITERATURE REVIEW

Xi Qin, Wei Wang, *et.al* (2016) In order to prevent progressive collapse, this study discusses research on two different forms of beam-to-tubular column connections. Two examples were built to depict these two different sorts of connections, each consisting of a central column stub and half of the beam span on either side. The ST-WBR specimen, which was altered using a unique reinforcing approach based on the connection detail of the ST-WB specimen, had a reinforced welded flange-bolted web connection. One specimen (ST-WB) had a normal welded flange-bolted web connection.

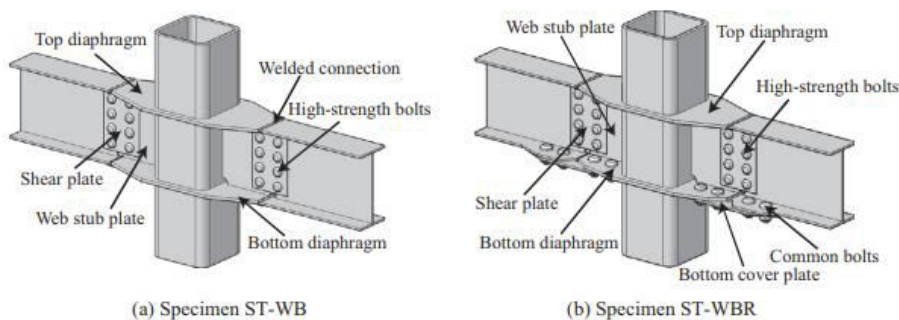


Figure 2: Specially Reinforced Beam Column Connection

To study the specimens' behaviours in a scenario where the central column is removed, the vertical displacement of the central column was increased monotonically. A vertical reaction frame mounted on the strong floor supported the vertical load that was applied to the top of the central column by an actuator with a 2000 kN capacity. About 450 mm was the highest vertical displacement range. During the testing, load was delivered using a displacement control strategy at a speed of less than 7 mm/min. When a collapse mechanism formed for each specimen and the vertical load-carrying capacity was exhausted, the test was declared complete. Detailed finite element models were used to simulate the tests computationally. The analyses provide perceptions on the actions and modes of failure of the connections, as well as how well they handled the tensile tensions that built up in the beams. The reinforced welded flange-bolted web connection may make the bottom flange continue to transfer force effectively after the weld failure, which is advantageous to the ongoing development of axial tensile forces in the beams, according to both experimental and computational results. Both specimens initially react similarly to an increase in vertical displacement at the central column. Before the bottom welded connection failed, the specimens had the same degree of vertical displacement. The

two specimens' maximum vertical loads are more than FP, which clearly demonstrates that the effective catenary action as well as the flexural action had formed in the specimens under the conditions of an internal column removal scenario. After the bottom welded connection failed, two specimens showed two different reactions. When comparing ST-WB to ST-WBR, ST-WBR's maximum vertical load increased by 134% in comparison to ST-WB's. It is clear that the unique reinforcing approach significantly improves vertical load carrying capacity.

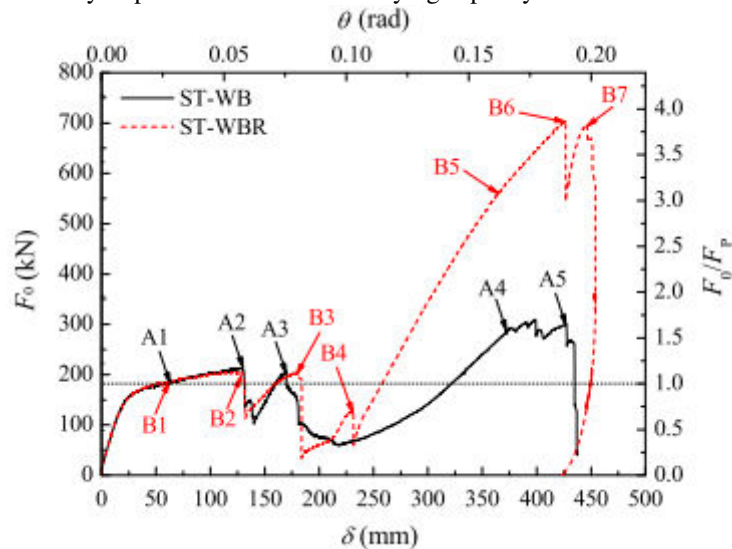
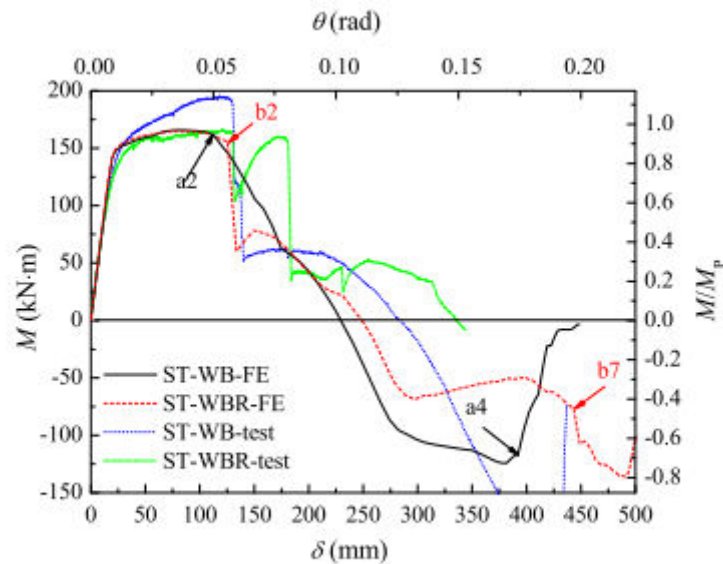


Figure 3: Vertical Load Resisted w.r.t Displacement



(b) Bending moment.

Figure 4: Bending Moment w.r.t Displacement

Contributions to the vertical resistance by flexural and catenary action: In the case of removing the central column, the axial tension and shear force of the connecting beams will carry the vertical load F_v that was before borne by the removed Column. Resistance force from catenary action F_c and resistance force from flexural action F_f are defined by the following equations for a quick estimation of the contributions from the two forms of force.

$$F_c = N_a \sin \theta + N_b \sin \theta$$

$$F_f = F_v - F_c$$

Where N_A and N_B stand for the two pin supports' respective axial reaction forces as determined by FE findings. Figure shows the evolution of F_c and F_f , where F_c and F_f are normalised in relation to F_p .

Conclusion: The assembly with reinforced welded flange-bolted web connection has a vertical load carrying capacity that is more than twice as high as the assembly with conventional welded flange-bolted web connection. Consequently, the unique strengthening method suggested in this work can be used to construct frame structures to prevent progressive collapsing.

Canwen Chen *et.al* (2020) In this work, nine laboratory experiments were carried out to calculate the performance of reduced beam section (RBS) connections, which are earthquake-resistant structures, in the event that the centre column is destroyed. To examine the impact of RBS and various beam web connection techniques, four types of joints were tested, including welded unreinforced flange - bolted web connection without RBS (WUF-B-NRBS), welded unreinforced flange - bolted web connection with RBS (WUF-B-RBS), and welded unreinforced flange - welded web connection without RBS (WUF-W-NRBS).

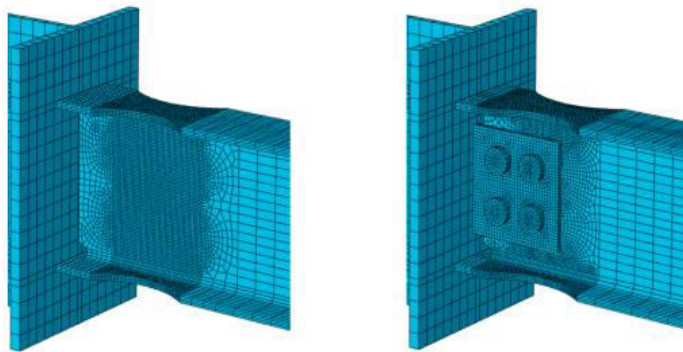
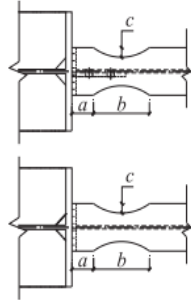


Figure 5. Welded flange - welded web and welded flange - bolted web

Table 1.

Specimen data.				
Specimen ID	Beam web connection method	<i>a</i>	<i>b</i>	<i>c</i>
WUF-B-NRBS	Bolt web	-	-	-
WUF-B-RBS-50-130-23	Bolt web	50	130	23
WUF-B-RBS-50-170-23	Bolt web	50	170	23
WUF-B-RBS-75-130-23	Bolt web	75	130	23
WUF-B-RBS-75-130-25	Bolt web	75	130	25
WUF-W-RBS-50-130-23				
WUF-W-NRBS	Welded web	-	-	-
WUF-W-RBS-50-170-23	Welded web	50	130	23
WUF-W-RBS-50-170-23	Welded web	50	170	23
WUF-W-RBS-75-130-23	Welded web	75	130	23



General behaviours and resistance mechanism: After the middle column is removed, the shear force and axial force, which are provided by the flexural mechanism and catenary mechanism, respectively, resist the applied load on the middle beam-to-column joints. The internal force can be computed using the reduced computational model by measuring the strain in the projected elasticity section. Equation (1) to (4) can be used to determine the resistance mechanism.

$$N_{L1} = EA \left(\frac{\sum_{i=1}^m \epsilon_i}{m} \right) \tag{1}$$

$$N_{R1} = EA \left(\frac{\sum_{j=1}^n \epsilon_j}{n} \right) \tag{2}$$

$$V_{R-C} = N_{L1} \sin \theta_{L1} + N_{R1} \sin \theta_{R1} \tag{3}$$

$$V_{R-F} = P - V_{R-C} \tag{4}$$

Where E is the elastic modulus, and NL1 and NR1 stand for the axial forces of elastic sections L1 and R1, respectively. m and n are the number of strain gauges in sections L1 and R1, respectively; L1 and R1 are the rotation angles of sections L1 and R1, respectively; and A is the sectional area of the specified section in the beam end. P stands for the load applied to the central column, VR-C for the load resisted by catenary action, VR-F for the load resisted by flexural action.

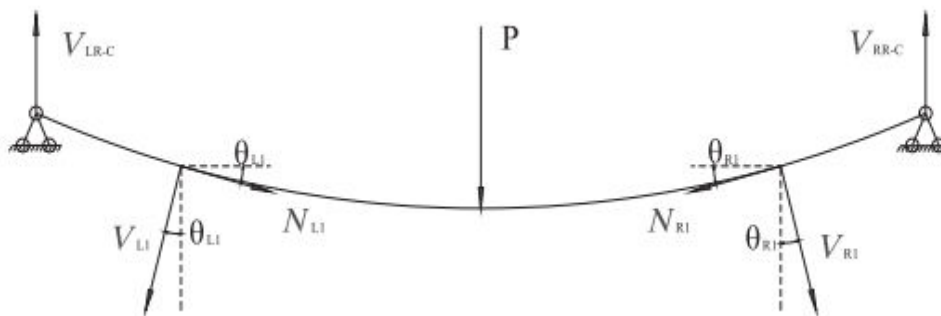


Figure 6: Free Body Diagram

Conclusion: In this study, nine specimens made up of four different types of connection joints were examined to examine the general behaviour, resistance mechanism, and mode of failure of steel moment frames in the event that the centre column is removed. The influence of RBS, beam web connection techniques, and weakening parameters were examined in accordance with the comparison of various specimens, connections, and connection performances.

Wenda Wang *et.al* (2017) By comparing the findings of the nonlinear static analysis and the nonlinear dynamic analysis, it was possible to determine the link between the resistance capacity and resistance demand of these joints. The analysis's findings demonstrated that the frame construction with these joints, which made it possible to create a resistance mechanism and a new alternate path for imbalanced loads, can avoid progressive collapse in the event that a column connected to the joints fails.

Nonlinear static analysis: Progressive collapse is a nonlinear dynamic process, however nonlinear static analysis can clearly show the failure mode and resistance of progressive collapse. Therefore, the steel beam to CFST column junctions were first investigated using the nonlinear static method. The middle column was first eliminated from the nonlinear static analysis, and then the vertical displacement U was applied to the base of the remaining column that was connected to the middle joint until the middle joint lost its resistance. The failure mechanisms and anti-progressive collapse process of these joints might be discovered using the nonlinear static pushdown.

The results of the nonlinear static analysis show a similar trend in the resistance mechanisms of these four joints. The cross profiles of steel beams are selected as the analysis region in order to examine the characteristics of resistance mechanisms of these junctions. The axial direction stress S11 distribution in the cross profile of the steel beams is shown in Fig.7 using the SJ-O as an example. The beam mechanism phase is depicted in Fig.7a. The lower flange of the beam section is in tension during this phase, while the upper flange is compressed. The neutral axis is almost in the

centre of the web. The mixed mechanism phase is seen in Fig.7b. The neutral axis rises with the middle joint's vertical motion. The cross profiles tension areas increasingly grow larger while the compression zones shrink. Phase of the catenary mechanism is depicted in Fig7c. The steel beam's entire segment is under tension. Figure depicts the failure phase. When the bottom flange and the web begin to break down, the stress in these areas is zero, but the upper flange retains some tension and resistance-building capacity. Until these joints break and are unable to produce any resistance, the resistance mechanisms that are provided by them alternate between the catenary and beam resistance mechanisms. Fig7d. Includes the anti-progressive collapse mechanisms of these joints. The anti-bend capacity of steel beams is represented by the OA phase as the beam mechanism. The BC phase, which stands for the catenary mechanism, is characterised by stress across the entire cross section of the steel beam, which is made possible by the steel beams' tensile strength. Between the OA and BC phases comes the AB phase, which denotes a hybrid mechanism. The beam mechanism gradually diminishes and the catenary mechanism gradually improves throughout the phase of mixed mechanisms. Failure is the label given to the CD phase because during this phase the steel beams begin to crack and the joints gradually lose their ability to prevent collapse.

Point A in Figure 8 represents the resistance of the beam mechanism phase. Steel beam sections' tension zones grow when they cross this point. The situation where the steel beam's overall cross profile starts to become tensed is represented by point B. The C-point on a steel beam serves as the initial point of fracture and displays the resistance of the catenary mechanism phase. The outcome demonstrates that the catenary system may enhance the ability to prevent collapse of joints to varying degrees. The frame structure could provide additional support if the catenary mechanism's development fully exploits the resistance to progressive collapse.

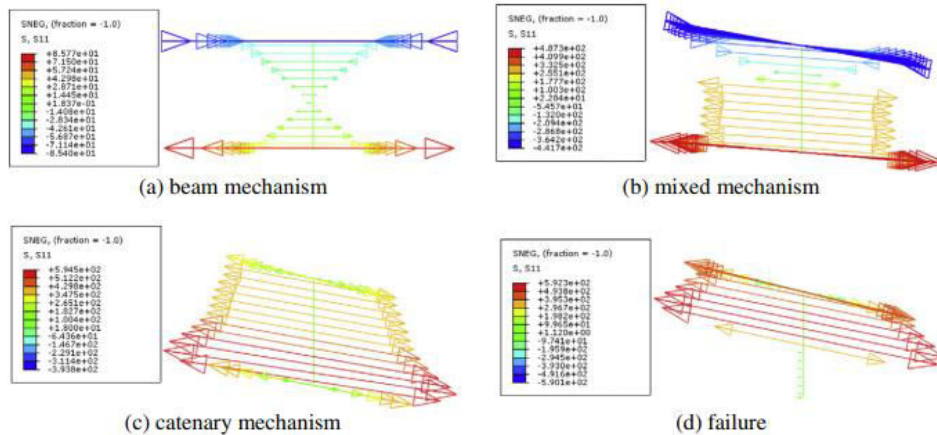


Figure 7: Axial Direction Stress Distribution S11

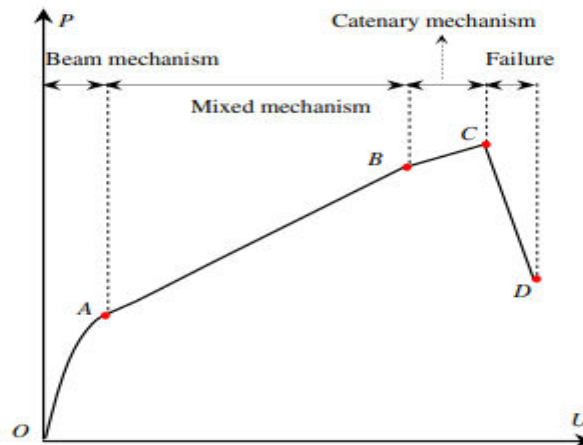


Figure 8: Anti Progressive Collapse Mechanism

Conclusion: This study used a multi-scale model to analyse the progressive collapse of the connections between the steel beam and the CFST column. The authors conducted analysis in these studies using both the nonlinear static approach and the nonlinear dynamic method. From this paper, several inferences can be drawn.

Long Zeng et.al (2022) In this paper, an experimental and numerical study was carried out on the anti-progressive collapse performance of the fabricated connection with CFST column and composite beam. Three types of fabricated connections and two types of shear connectors were compared to investigate the failure mode, vertical resistance-deformation mechanism, and strain-deformation relationship. The bending moment and axial force of the steel beam as well as the contribution ratio of each component to vertical resistance were calculated and analysed. Moreover, the nonlinear parameters of progressive collapse resistance of the specimens were compared with DOD and FEMA350 criteria. Finally, the dynamic increasing factors (DIFs) by the test, finite element (FE) model, and energy balanced (EB) method were discussed.

Contribution of each component to vertical resistance: The FA and CA of the steel beam and the RC slab, respectively, provided the majority of the vertical resistance against progressive collapse (F) for the CFST column-to-composite beam connection. According to Fig. 9, Eqs. (3), (4), and (5) can be used to calculate the vertical resistance offered by the FA of steel beam (F_{FA}), the CA of steel beam (F_{CA}), and the RC slab (F_{RCS}), respectively.

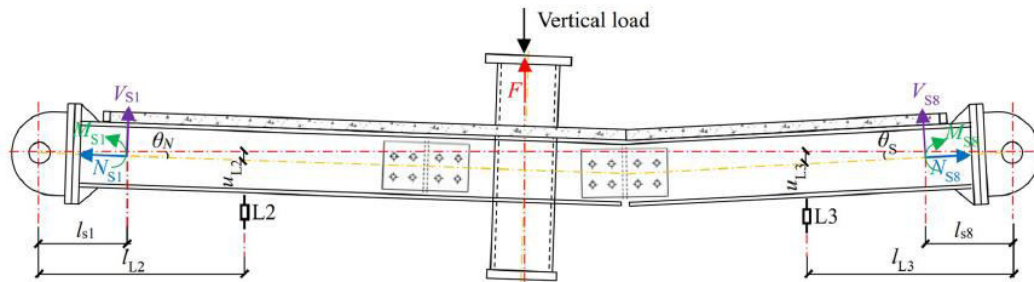


Figure 9: Contribution of Catenary and Flexural Action of steel beam to vertical resistance

$$F_{FA} = V_{S1} \cos \theta_N + V_{S8} \cos \theta_S \tag{3}$$

$$F_{CA} = N_{S1} \sin \theta_N + N_{S8} \sin \theta_S \tag{4}$$

$$F_{RCS} = F - (F_{FA} + F_{CA}) \tag{5}$$

Where, according to Eq. (6), V_{S1} and V_{S8} represent the shear forces of sections $S1$ and $S8$, respectively. The chord rotation of the beams on the north and south sides, respectively, is given by the expression in Eq. (7) as N and S .

$$V_{Sn} = \frac{M_{Sn}}{l_{Sn}} \tag{6}$$

$$\theta_N = \frac{u_{L2}}{l_{L2}}; \theta_S = \frac{u_{L3}}{l_{L3}} \tag{7}$$

Where V_{Sn} and M_{Sn} stand for the section S_n 's shear force and bending moment, respectively. Section S_n 's distance from the hinge support is represented by the number l_{Sn} . The vertical deformations determined by the LVDTs $L2$ and $L3$, respectively, are designated as u_{L2} and u_{L3} . The distances between the hinge supports at the north and south sides and the LVDTs $L2$ and $L3$, respectively, are l_{L2} and l_{L3} .



Table 2:
Contribution ratios of components to vertical resistance.

Specimen	First peak (FP) vertical resistance			Second peak (SP) vertical resistance		
	FA of steel beam	CA of steel beam	RC slab effect	FA of steel beam	CA of steel beam	RC slab effect
CJ-WB	88.7%	11.3%	/	/	/	/
CJ-B	33.8%	64.2%	/	3.0%	97.0%	/
CJ-WB-RCS	68.1%	7.4%	24.5%	/	/	/
CJ-WB-RCS-PBL	29.7%	8.2%	63.1%	/	/	/
CJ-RWB-RCS	45.9%	14.2%	39.9%	/	/	/
CJ-B-RCS	14.9%	40.4%	44.7%	-28.5%	107.7%	20.8%

Dynamic increasing factor (DIF) analysis: The dynamic increasing factor (DIF) is advised by the Department of Defence for evaluating anti-progressive collapse performance in the event of a sudden column loss. The static load-vertical deformation curve and the dynamic load-max deformation curve from FE modelling or pseudo-static simulations can both be used to determine the DIF-normalized rotation.

$$DIF = \frac{F_{s,n}}{P} = \frac{F_{s,n}}{F_{p,n}} \tag{10}$$

$$\theta_n = \frac{\theta_p}{\theta_y} = \frac{\theta - \theta_y}{\theta_y} \tag{11}$$

Where θ_n and θ_p , are normalized rotation and plastic rotation, θ_y can be determined as follows.

$$\theta_y = \frac{Zf_y l_b}{6E_s I_b} \tag{12}$$

Where Z is plastic section modulus, l_b is beam length, and I_b is the moment of inertia of beam.

Conclusion: From this study researcher stated that the relation between static load and dynamic load plays very important role in performance of connection.

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Load and Dynamic Increase Factors.

The three analytical techniques of linear static, nonlinear static, and nonlinear dynamic may be used. Progressive collapse is a dynamic and nonlinear event; therefore, load increase factors or dynamic increase factors, which roughly account for inertial and nonlinear effects, must be used in the applied load situations for the static processes. The 2005 UFC 4-023-03 and the GSA Guidelines use a load



multiplier of 2.0, directly applied to the progressive collapse load combination for both linear static and nonlinear static.

Figure 10: illustrates the outcomes of this procedure for the Dynamic Increase Factor (DIF), which is used for nonlinear static analyses of steel structures. The DIF is displayed as a function of the normalized rotation, which is the allowable plastic rotation divided by the rotation at yield of the cross section. The data points in this figure were produced by examining a variety of structures with different heights, bay sizes, and structural characteristics. With this figure, the designer has the option of selecting the DIF based on the level of nonlinear behaviour (i.e., structural performance level) they choose to use, or else they can assign the degree of nonlinear behaviour and get a specific DIF as a result. The designer will utilize this value to determine the DIF from the suggested equation in Figure by finding the structural component or connection with the smallest normalized rotation for any area of the structure that will be affected by the removal of the column.

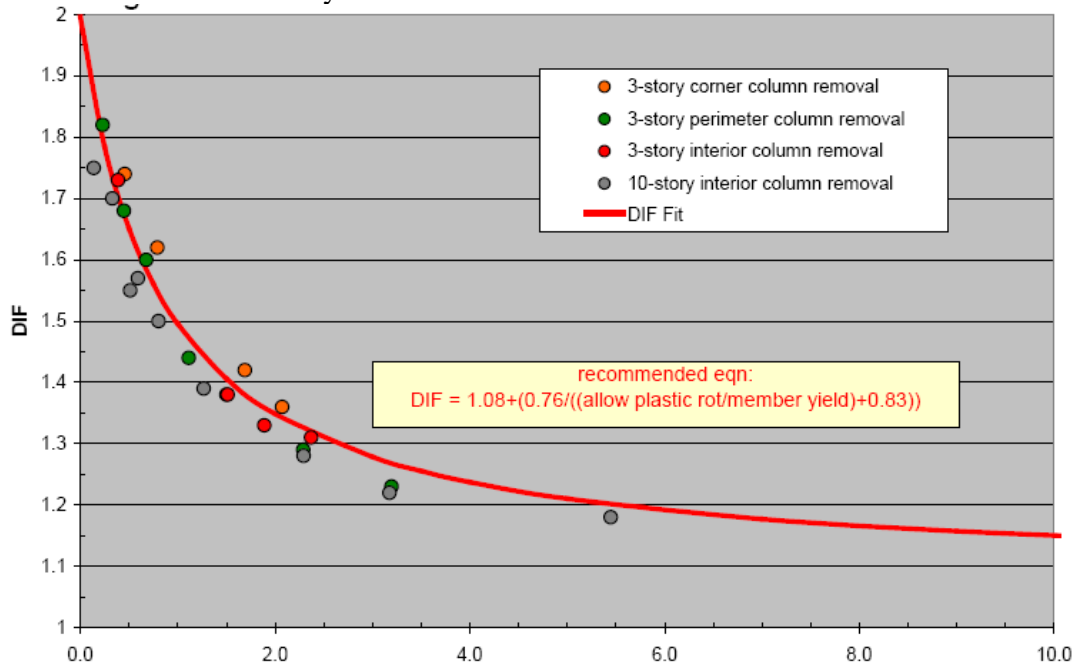


Figure 10: Dynamic Increasing Factor for Structural Steel

III. STUDY

Numerous elements must be taken into account while designing an anti-collapse beam column connection. Numerous studies have been conducted, and it has been found that the beam column connection with the diaphragm performs best of all, yet loading is still done on it. However, because progressive collapse is a dynamic, nonlinear occurrence, the link must offer adequate resistance. Both catenary movement and flexural action can cause this resistance. The most effective approach to transmit load to prevent Progressive collapse is through catenary action. The analysis of the beam-column connection against dynamic loading under progressive collapse should not be limited to maximizing resistance against vertical loads.

The relationship must be examined by taking into account hysteresis behavior, energy dissipation, maximum load achieved, stress development and anti-progressive collapse mechanism, flexural action, catenary action, vertical reactions developed at support, and dynamic increasing factor by studying various non-linear static, pseudo-static, and dynamic loading researches.

IV. CONCLUSION

This study simply demonstrates that the behaviour of a beam-column connection under static and dynamic loading conditions can be investigated. All of these parameters and methods for achieving the parameters have been extensively examined in this work.



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